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Braking test from high speed at Andrews Air Force Base, Maryland

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IN THIS ISSUE

U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
FRANCIS V. du PONT, Commissioner

Braking Distances of Vehicles from High Speed

and Tests of Friction Coefficients

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
O. K. NORMANN
Chief, Traffic Operations Section

The limited tests to examine braking distances of vehicles from high speed and to measure coefficients of friction, reported in this article, were of sufficient scope to throw doubt on some of the beliefs heretofore commonly accepted.

What braking force or deceleration rate should be expected? Are stopping distances from high speeds longer than generally accepted as correct? What causes the wide variation in braking action of different vehicles, and of the same vehicle in different trials? Are deceleration rates attainable by vehicles at high speeds uncomfortable to the passengers, so long as the vehicle follows a straight path? To what extent do brake fade and other factors affect stopping distances?

Tentative indications as to the answers to these questions were found in the braking tests described here, but they serve principally to point the need for far more extensive studies that should involve the cooperation of the automotive and tire industries.

The results of the friction coefficient measurements that were made are useful as a pilot study to illustrate the required magnitude of any investigation of the interrelation of stopping distances and friction coefficients. It appears necessary to consider variation in the nonskid qualities of both tires and road surfaces to obtain the most effective improvement in operating safety.

THE late Ernest E. Wilson, at that time director of the General Motors proving ground, reported in December 1940 the results of tests to determine deceleration distances for high-speed vehicles. He used 15 passenger cars in perfect condition and 8 highly experienced test drivers to obtain braking distances for speeds ranging from 50 to 70 miles per hour. Since 1940, about 35 million passenger cars have been built in the United States, but few if any results of tests to determine braking distances from high speeds have been reported. There have been numerous reports for tests made from speeds of 20 to 40 miles per hour, but passenger cars are now being operated at an average speed of about 52 miles per hour on our main rural highways, with about 12 percent exceeding 60 miles per hour, and an occasional vehicle traveling in excess of 80 miles per hour.

One possible reason that more high-speed tests have not been conducted is the common assumption that any passenger car with brakes in good condition can lock all four wheels and that shorter stopping distances can be realized only through improving the texture of road surfaces to obtain higher coefficients of friction, especially for wet surfaces, and by avoiding the use of tires that have worn smooth. Although a more critical skidding condition usually exists when a surface is wet than when dry, about three out of four fatal accidents and two out of three of all accidents occur on dry surfaces. Many of these no doubt could have been avoided if the drivers had been able to stop a little sooner.

To obtain current information on braking distances from high speeds, the Bureau of Public Roads in 1949 conducted a series of tests on 53 vehicles representing 10 of the most common makes. Twelve of the cars were Government vehicles owned by the Bureau of Public Roads. The other 41 were private cars owned by employees of the Bureau of Public Roads who volunteered to participate in the tests. Each vehicle was driven during the test by the person who normally operated the vehicle.

Stops similar to those that would have been made in an emergency were made by all of the drivers from speeds of 20 and 40 miles per hour. Most drivers also made emergency stops from 60 miles per hour, and the drivers of the Government vehicles and some of the private cars made stops from the highest attainable speed, which was generally over 70 miles per hour. Data were obtained for a total of 214 stops, including 14 at speeds exceeding 75 miles per hour and 7 at 90 miles per hour.

Test Procedure

The tests were conducted on a concrete taxiway 6,500 feet long and 50 feet wide at Andrews Air Force Base. The surface had a broomed finish and was free of oil drippings since it had been used very little. About 2,000 feet from the end of the taxiway, a rubber tube with an air-switch on one end was stretched across the surface. As the front wheels of a vehicle passed over this tube, a solenoid was actuated which turned on a brilliant light 700 feet away on the right-hand side of the surface. This light acted as a signal to inform the driver to apply his brakes as quickly as possible. Immediately ahead of the tube which actuated the light were three other tubes connected to a recording speedmeter located in a panel truck parked at one side of the surface. Accurate speeds could thus be obtained immediately prior to the brake application for vehicles traveling within the range of 15 to 100 miles per hour.

Each vehicle to be tested was equipped with a detonator mounted on the front bumper (fig. 1) and actuated electrically through a switch on the brake pedal (fig. 2). When the brake pedal was touched, the detonator fired a .22-caliber cartridge, ex-

Figure 1.—Mounting detonator on vehicle.

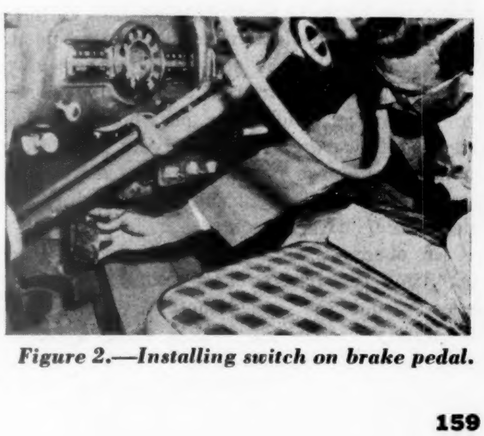

Figure 2.—Installing switch on brake pedal.

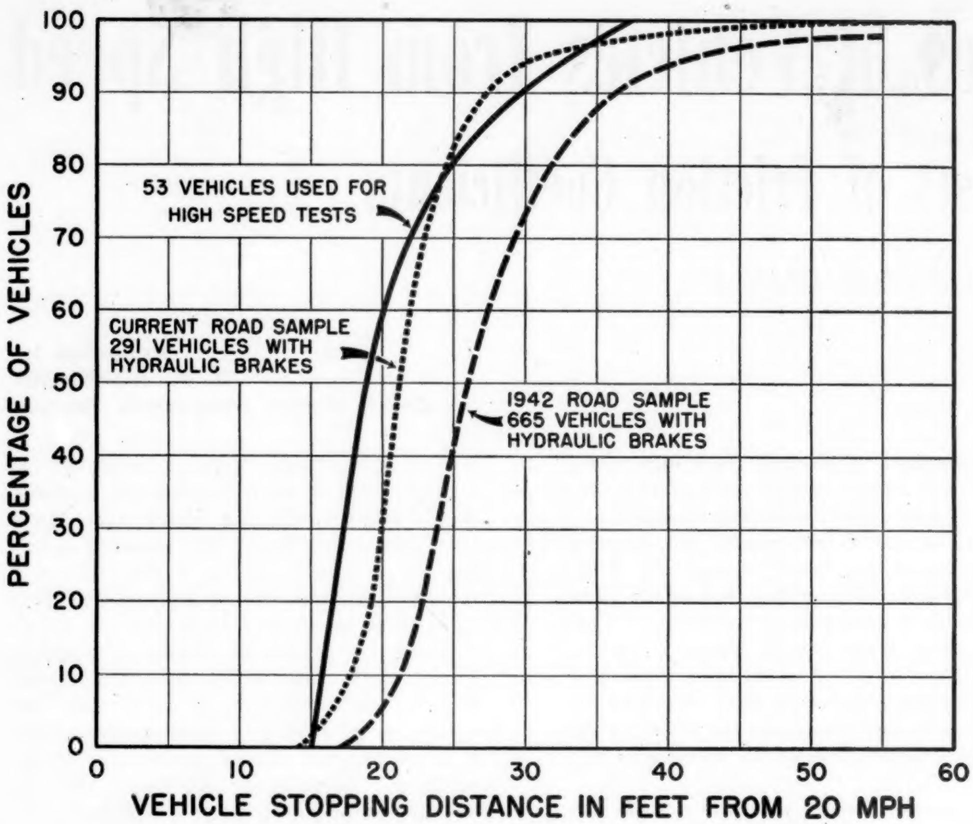

Figure 3.—Stopping distances for various groups of vehicles sampled.

pelling a pigment-filled capsule which left a yellow mark 5 inches in diameter on the pavement to indicate the point at which the brakes were first applied. The distance between the tube which actuated the light and the yellow mark was the distance traveled during the driver's reaction time, and the distance between the yellow mark and the point where the vehicle came to a stop was the braking distance. Measurements were made by use of a scale marked directly on the concrete surface.

Prior to testing each vehicle, the following information was recorded: Year model and make of vehicle, present mileage on vehicle, mileage when brakes were last relined or adjusted, distance between floor boards and brake pedal when depressed, condition of tires, and name and age of driver.

During each brake test, observers stationed along the taxiway noted whether any wheels skidded and, if so, which wheels locked. The drivers were requested to be in high gear before reaching a speed of 20 miles per hour, to accelerate as fast as possible to the predetermined braking speed, to continue at that speed until they saw the spotlight illuminated, and then to come to a stop as quickly as possible. No instructions were given the drivers regarding use of the clutch pedal and no observations were made as to when the clutch pedal was depressed during the stops.

An observer riding in the vehicle noted the time required to accelerate to various speeds and the speedometer reading immediately before the brakes were applied. The recorded speedometer readings were later adjusted to conform with actual speeds. The observer also kept the driver informed of the speedometer reading so that the driver could keep his eyes trained on the road and spotlight. After the test, the observer questioned the driver as to whether he considered the stop a comfortable one, whether he thought the stop could have been made in a shorter distance, and whether he applied his maximum force to the brake pedal during the stop. The observer also measured the distance between the floor board and the depressed brake pedal.

Performance Better Than Average

Only 12 of the 53 vehicles used for the high-speed brake tests were new vehicles; 3 were 10 years old. The average age of the 53 test vehicles was 3.4 years, less than half the average age of 7 years for all passenger cars registered in the United States during 1951. All of the drivers participating in the test considered their brakes in good condition, and it will be shown that the vehicles participating in the brake tests could be stopped from a speed of 20 miles per hour in shorter distances than the average vehicle being operated on public highways.

Comparisons of stopping distances from a speed of 20 miles per hour for the 53 vehicles used in the high-speed tests with corresponding values for vehicles included in other comprehensive studies are shown in figure 3. The results for the 1942 road sample and the current road sample were obtained by conducting brake tests on vehicles selected at random from everyday traffic on main rural highways. Each vehicle selected at random was subjected to three or more emergency stops from 20 miles per hour on dry concrete pavements.

It may be noted from figure 3 that there has been a substantial improvement since 1942 in the brake performance of vehicles in operation on our highways. In 1942, only 40 percent could be stopped in less than 25 feet and 13 percent required more than 35 feet. Today, 83 percent can be stopped in 25 feet and only 3 percent require more than 35 feet. There are still, however, some vehicles being operated on our highways with brakes in such poor condition that they cannot be stopped from 20 miles per hour in 60 feet, more than four times the distance required for some vehicles.

None of the vehicles used for the high-speed tests required more than 37 feet to stop from 20 miles per hour. On an average, they showed somewhat better brake performance from 20 miles per hour than those selected for the current road sample. This was to be expected, since most of the drivers who volunteered for the high-speed tests thought, no doubt, that they and their vehicles would perform as well or better than average. Most of the drivers undoubtedly volunteered because they were willing to spend some time and money in helping to obtain useful facts relating to

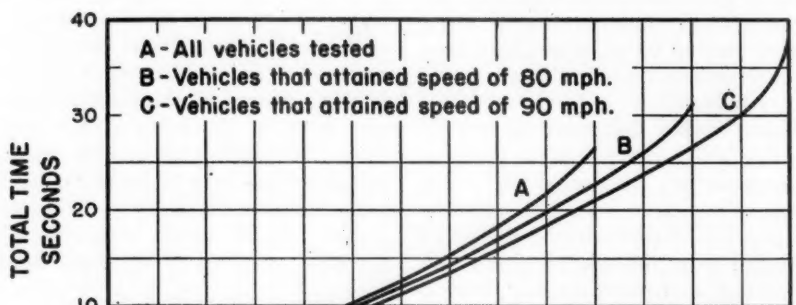

Figure 4.—Time to accelerate from 20 miles per hour in high gear.

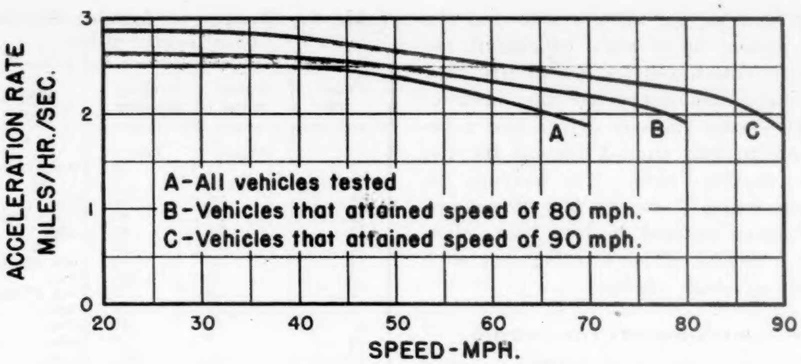

Figure 5.—Average acceleration rate from 20 miles per hour.

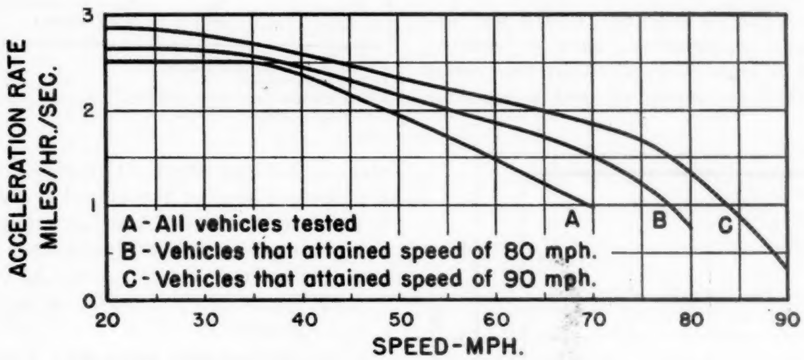

Figure 6.—Instantaneous acceleration rates at various speeds.

one of the most necessary elements of highway safety, the ability of vehicles to stop from high speeds. It is doubtful, however, that any one of the drivers would have participated in the test had he known that his performance would have been the worst of the lot.

Acceleration Rates

Before discussing the more involved results of the brake-performance tests, it seems desirable to dispose of a few of the results of these tests that may be termed byproducts. These relate to the acceleration rates of passenger cars, reaction times of drivers, and accuracy of speedometers. Figures 4, 5, and 6 relate to the acceleration rates of passenger cars. In these figures, the passenger cars have been classified into three groups. Group A includes all of the vehicles tested; group B includes only the vehicles that reached a top speed of about 80 miles per hour on the test course; and group C includes only the vehicles that reached a top speed of 90 miles per hour on the test course.

Figure 4 shows the total time required to accelerate to any given speed from a speed of 20 miles per hour, in high gear. It is also possible to determine, by subtraction, the time to accelerate from a given speed to any higher speed within the range of the chart. The vehicles capable of a 90-mile-per-hour speed (group C) could accelerate from 20 to nearly 70 miles per hour in approximately the same time as they required to accelerate from 70 to 90 miles per hour. They traveled an average of 1,450 feet in going from 20 to 70 miles per hour, and 1,850 feet in going from 70 to 90 miles per hour.

The average acceleration rates while increasing from a speed of 20 miles per hour to a higher speed are shown in figure 5. The average rate of acceleration decreases as the speed increases, although the difference is slight between 20 and 40 miles per hour.

The instantaneous acceleration rates at various speeds are shown in figure 6. The average vehicle (group A) had an acceleration rate of 2.5 miles per hour per second at speeds between 20 and 35 miles per hour, whereas the vehicles that could reach 90 miles per hour (group C) had a maximum instantaneous acceleration rate of 2.8 miles per hour per second at a speed of 20 miles per hour. Rates of 2.5 and 2.8 miles per hour per second are equivalent to 3.7 and 4.1 feet per second per second, respectively. As the speed approaches the top speed of a car, the acceleration rate approaches zero. From the curves of figure 6 it appears that the cars which reached a speed of 90 miles per hour could have eventually reached about 92 miles per hour, and those that reached a speed of 80 miles per hour could eventually have increased their speeds to 85 miles per hour.

Information regarding the acceleration rates of passenger cars is used in connection with many highway design problems relating to traffic operations. In recent years the principal source of such information has been the results obtained for six passenger cars tested in 1938. The results for the 53 cars in current use indicate that acceleration rates at the present time are 20 to 30 percent higher than for the six cars tested in 1938.

Driver Reaction Time

The measurements of driver reaction time and distance that were made during the braking-distance tests from high speeds may be considered as absolute minimums. Each driver was aware of the approximate time that the stop was to be made and was poised for the occasion. Only the reaction time for the initial test run of each driver has been used for this analysis. During subsequent tests, several of the drivers anticipated the time that the signal light would be illuminated and had removed their foot from the accelerator prior to reaching the road tube that actuated the light.

Figure 7 shows the distribution of reaction times for the 53 drivers. Some drivers

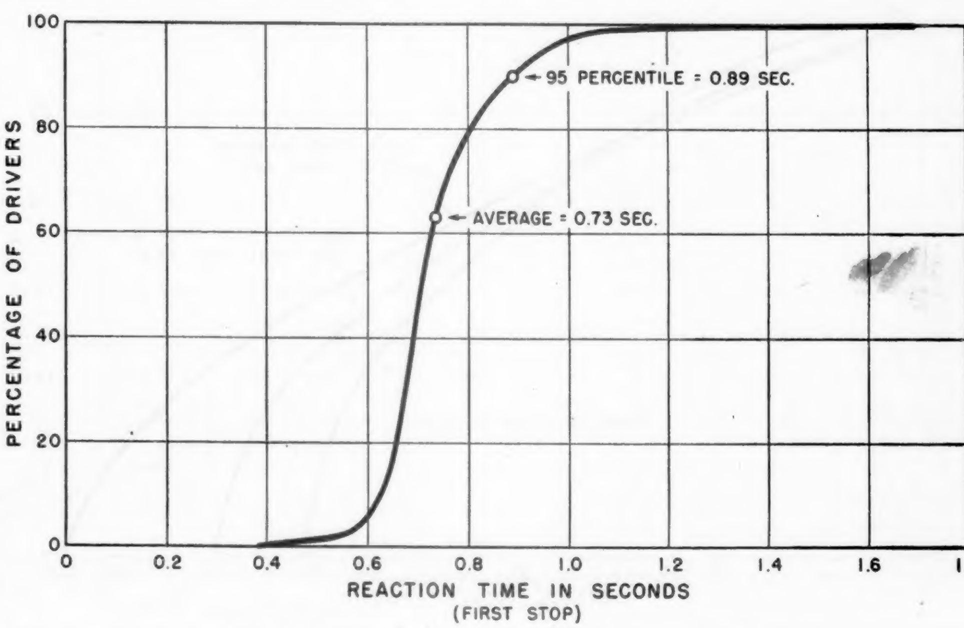

Figure 7.—Reaction times during high-speed tests.

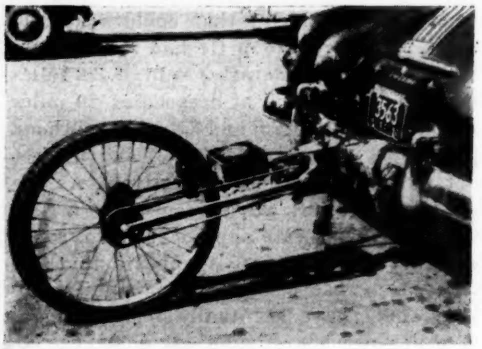

Figure 8.—Fifth wheel used to obtain time-distance curves.

reacted and moved their right foot from the accelerator to the brake pedal in 0.4 second. One driver required 1.7 seconds. Repeated tests on the drivers who had the longer reaction times gave consistent results which eliminated the possibility that these drivers misinterpreted the instructions. All except the one driver had a reaction time of less than 1 second for the conditions of these tests. The average reaction time was 0.73 second and 95 percent of the drivers reacted in less than nine-tenths of a second, which is consistent with the results of other studies.

Most Speedometers Inaccurate

It was interesting to find that more of the speedometers were low than high when compared with the actual speeds of the vehicles. This is contrary to the common belief that speedometers have a tendency to indicate higher speeds than the actual speeds. If it is considered that a speedometer is correct when it registers within 2 miles per hour of the actual speed, which is as close as the readings on many of the present models can be determined, the following were the results for the 53 vehicles:

19 speedometers correct at all speeds tested.

6 speedometers more than 2 miles per hour high.

28 speedometers more than 2 miles per hour low.

The average error for the 6 speedometers that were high was 6.7 percent for speeds below 50 miles per hour and 6.1 percent for speeds above 50 miles per hour. At no speed did any of these speedometers have an error of more than 10 percent.

The average error for the 28 speedometers that were low was 12.1 percent for speeds under 50 miles per hour and 10.1 percent for speeds over 50. Four of the speedometers were between 20 and 24 percent low at all speeds: These indicated 57 to 60 miles per hour when the vehicles were actually traveling 75 miles per hour. The methods used for conducting the test eliminated the possibility that these errors resulted from a lag in the speedometers.

Table 1.—Results of braking-distance tests of a single vehicle.

Test No.	Initial speed	Braking distance	Skid marks
	M.p.h.	*Feet*	
1	18	14	Four wheels
6	19	14	Do.
16	30	48	None
12	30	45	Four wheels
11	30	46	Light
17	30	50	Do.
2	45	80	Four wheels
3	45	133	None
7	53	137	Four wheels [1]
8	53	162	None
13	79	540	None
14	78	468	None
15	80	564	None
18	80	400	Light [2]
19	81	400	Four wheels [3]
4	90	608	None
5	90	569	None
10	90	482	Four wheels [1]
9	90	640	None [4]

[1] Vehicle made a sharp dive.
[2] Very smooth stop.
[3] Four wheels locked for 251 feet.
[4] Attempt made to fan brakes.

Brake Fade

In all of the brake tests, actual measurement was made of the distance from the chalk mark on the pavement, indicating the point where the operator touched the brake pedal, to the point where the car came to rest. In addition, in a few of the tests a fifth wheel and chronograph were mounted on the test vehicle to obtain a time-distance record during each stop (fig. 8). From the time-distance record, deceleration rates were determined. Figure 9 shows a curve representing the average rates of deceleration during two stops from 30 miles per hour by one vehicle. In the one case, the stopping distance was 48 feet, and in the other it was 50 feet.

Figure 9 shows that the deceleration rate reached 22 feet per second per second within 0.6 second after the operator touched the

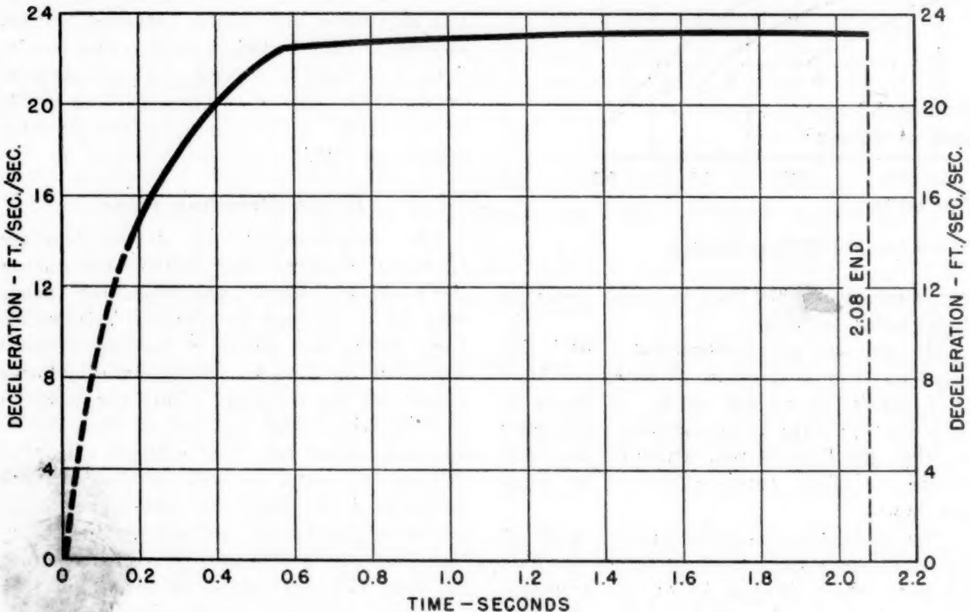

Figure 9.—Deceleration rates during stop from 30 miles per hour (average of two tests).

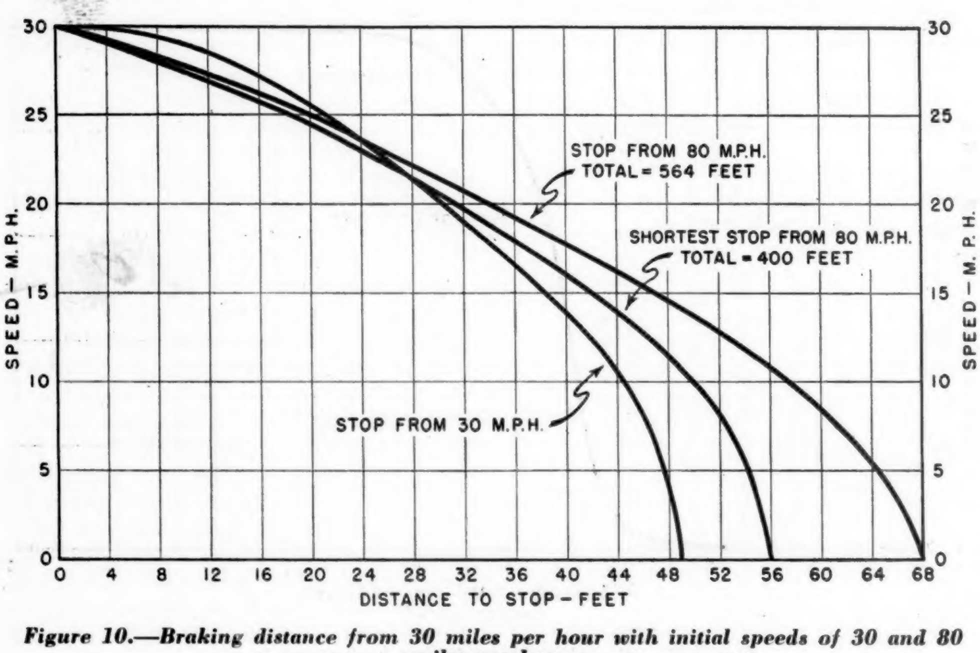

Figure 10.—Braking distance from 30 miles per hour with initial speeds of 30 and 80 miles per hour.

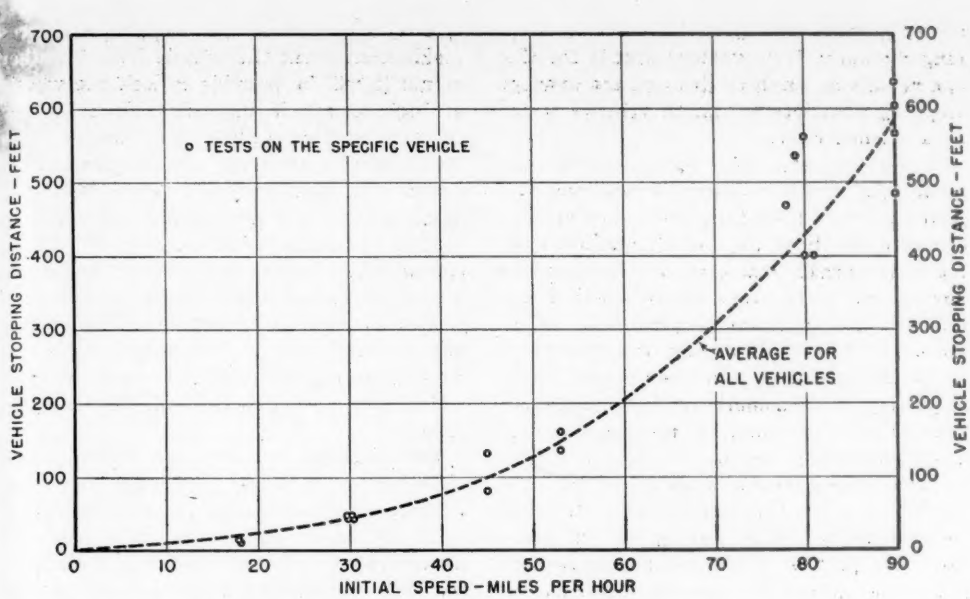

Figure 11.—Results of brake tests made on one specific vehicle as compared to all vehicles.

brake pedal. It is not known whether there was any deceleration within the first 0.1 second, but after 0.2 second had elapsed, the vehicle was decelerating at a rate of 15 feet per second per second. After 0.6 second there was only a very slight increase in the deceleration rate until the vehicle came to a stop.

During several of the stops from high speeds, the brakes seemed to fade shortly after being applied, making it appear as though the vehicle had very poor brakes. Brake fade may be defined as a temporary reduction in brake effectiveness resulting from heat. In such a case, the distance traveled to bring the vehicle to a stop after its speed had been reduced to 30 miles per hour seemed exceptionally long. In fact, one driver thought his brakes were not functioning during the latter part of a stop from 60 miles per hour. Fading usually did not appear to be pronounced during stops below 70 miles per hour except when several stops were made within a few

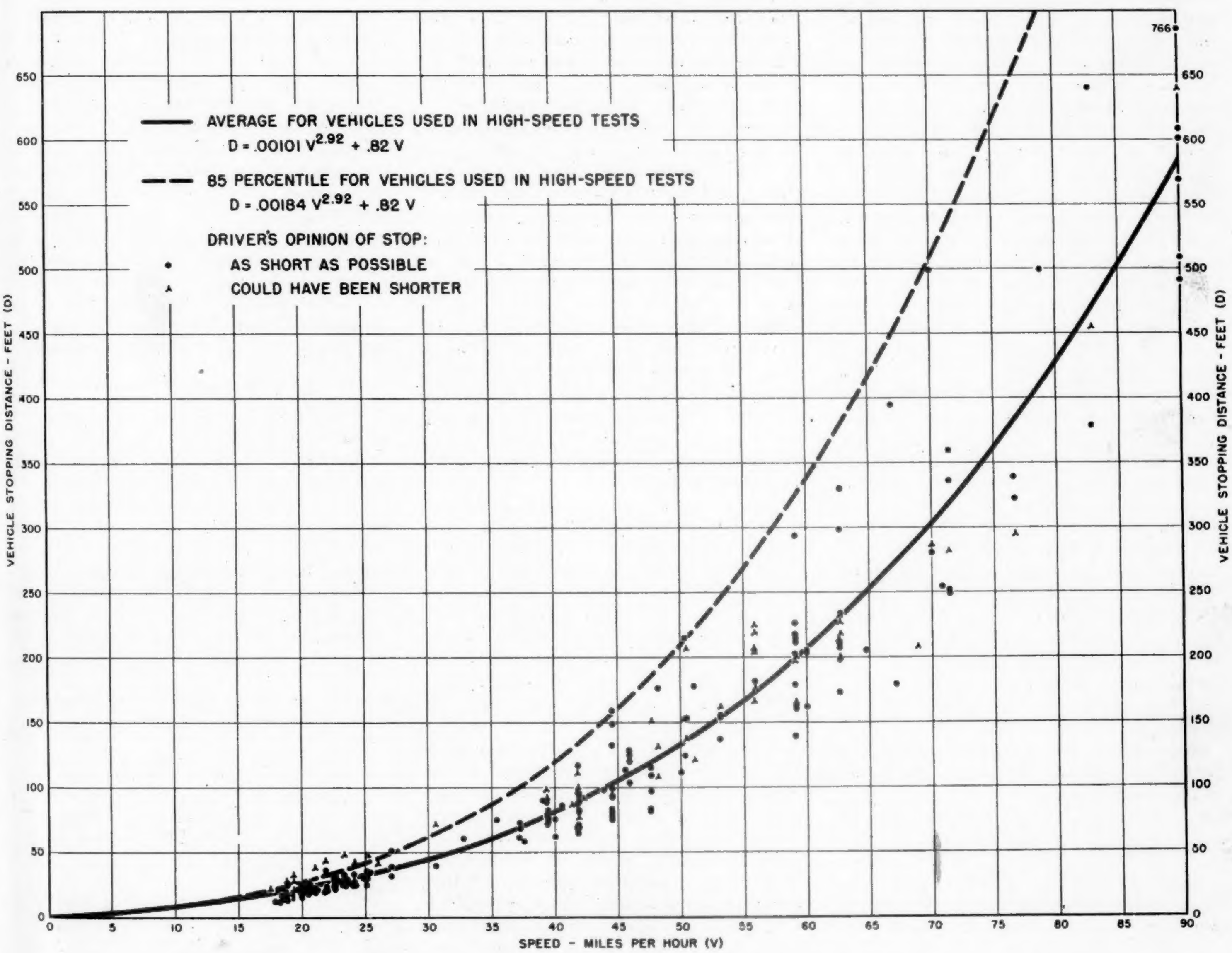

Figure 12.—Braking distances during high-speed tests.

minutes without giving the brake drums time to cool between the brake applications. The general procedure for these tests was to allow the brakes time to cool between successive applications.

The distances required to bring a particular vehicle to a stop from a speed of 30 miles per hour when the initial speed was 30 miles per hour in one case and 80 miles per hour in two other cases are shown in figure 10. The one stop from an initial speed of 80 miles per hour was made in 400 feet, the shortest distance in five tries, whereas the other stop from 80 miles per hour was made in 564 feet, the longest distance in five tries. None of the wheels locked during any of the three stops illustrated in figure 10, although the operator applied the maximum force possible to the brake pedal in each instance.

The speed-distance curve in figure 10 for the stop from an initial speed of 30 miles per hour is the average data for the same stops as those illustrated in figure 9. The curve in figure 10 shows that at the beginning of these stops there was a decrease in speed of only 1 mile per hour during the first 8 feet of travel. Shortly thereafter, the vehicle's speed decreased about 1 mile per hour for each 1.6 feet of travel, and immediately before coming to a stop the speed decreased 4 miles per hour while the vehicle traveled about 1 foot.

With an initial speed of 80 miles per hour, the vehicle was decelerating a little less than 1 mile per hour for each 4 feet of travel while going 30 miles per hour. During the remainder of the two stops, the speed decreased at a much lower rate for the same travel distance than during the stop made from an initial speed of 30 miles per hour.

One of the stops made from an initial speed of 80 miles per hour required 68 feet to bring the vehicle to a stop after the speed was reduced to 30 miles per hour, while in the other case the corresponding distance was 56 feet. The difference between these two distances and the 49 feet required for an initial speed of 30 miles per hour may be attributed to brake fade. In the one case the difference was 19 feet or 39 percent of the normal stopping distance for this vehicle from 30 miles per hour.

The tests on which the information in figure 10 was based were by no means the worst examples of brake fade that occurred during the series of tests. They were the only high-speed tests for which this type of data were obtained and have been presented to illustrate one reason that stopping distances do not vary exactly as the square of the speed.

Variation in Brake Performance

Under identical conditions, brake performance at high speeds was not always the same. The variation in braking distances for one vehicle with the same driver on the same surface is illustrated by the data shown in table 1. Figure 11 shows the stopping distances from various speeds for this one vehicle as small circles and the average stopping distances for all 53 vehicles tested as a dashed curve.

It may be seen from figure 11 that the results for this one vehicle, which was subjected to more tests than any other vehicle, compare closely to the average values. Fading of the brakes was especially pronounced during the three stops above the average line for speeds in the neighborhood of 80 miles per hour and during the two stops above the line at 90 miles per hour. These stops required considerably longer distances than other stops made by the same vehicle at corresponding speeds.

Table 1 also reports a record of the skid marks made by the test vehicle. It must be remembered that during all of these tests, except Nos. 9 and 19, the driver applied the maximum possible pressure to the brake pedal over the entire stopping distance. In some instances the wheels locked, leaving skid marks from all four wheels. In other instances none of the wheels locked. During some of the stops when the wheels did not lock, they were evidently not turning as fast as the car was traveling because light skid marks or tire traces were plainly visible, generally outlining the edges of the tire tread.

During the nine tests from speeds above 75 miles per hour, it was possible to lock the wheels only twice (Nos. 10 and 19). During test No. 9, an attempt was made to eliminate brake fade by fanning the brakes (removing all pressure from the brake pedal occasionally), but this apparently was not effective in reducing the total braking distance.

The results for tests Nos. 18 and 19 are especially significant. For both of these stops, which were made from approximately the same speed, the braking distance was 400 feet. Test No. 18 was a very comfortable stop, whereas the stop made during test No. 19 was the most dangerous one of the entire series.

During test No. 19, all four wheels locked at a point 31 feet beyond the point where the brakes were applied. The vehicle skidded for 151 feet, struck a construction joint in the pavement, and dived to the adjacent lane, at which time the brakes were released sufficiently to allow the wheels to turn. The vehicle then traveled 118 feet with the wheels turning and finally skidded 100 feet to a stop. The tires were so badly worn on one side as a result of this one stop that all four had to be replaced to eliminate a pronounced bumping as the vehicle was driven over a smooth surface.

The braking characteristics experienced during the 19 tests on this one vehicle indicate that if the wheels do not lock immediately after full pressure is applied, they cannot be locked at all. There evidently is enough heat developed with full brake pressure and the wheels turning so that fading soon takes place, especially at the higher speeds, and much longer stopping distances result than when the wheels lock. Whether or not it will be possible to lock the wheels at high speeds is unpredictable on a dry concrete surface of the type where these tests were conducted. Stops from high speeds with the wheels locked are, of course, dangerous from a viewpoint of maintaining control of the path of the vehicle. In an emergency, however, bringing the vehicle to a stop in the shortest distance possible may or may not be more important than the exact path of the vehicle, depending upon the circumstances. Even with the most careful and experienced drivers, such situations do arise.

The braking distance and initial speed recorded for each test conducted on all the vehicles are shown by the points in figure 12. In an attempt to determine the reason for the comparatively wide scatter of the points, the tests for which the drivers stated that they made the stop as fast as possible have been shown as dots, whereas the small triangular points represent tests for which the driver stated that he thought the stop could have been made in a shorter distance.

It should be noted that figure 12 shows a fairly even distribution of both types of points on both sides of the average curve. In fact, when the stopping distances that the drivers thought were not as short as possible were compared with those they thought were the shortest possible, it becomes evident that the driver's opinion is of little value. A summary of the results is shown in table 2. For speeds of 50 miles per hour or less, the braking distances for the stops made as fast as possible are slightly shorter than for the stops not made as fast as possible (table 2). For speeds above 50 miles per hour, the reverse is true, and the difference is more pronounced.

Comfort of Stop

It is also interesting to compare the actual braking distances with the drivers' opinions as to whether or not the stops could be considered comfortable. Invariably the more comfortable stops were made in the shorter distances even when stops by the same driver-vehicle combination were compared. Within the range of these studies it is therefore evident that other factors have a greater influence than the average deceleration rate on the comfort character-

Table 2.—Stopping distance related to driver opinion of performance

Initial speed	Stopping distance, when, in the driver's opinion, the stop—	
	Was made as fast as possible	Could have been made in a shorter distance
M.p.h.	*Feet*	*Feet*
20	25	34
40	86	90
50	126	146
60	210	206
70	311	259
80	464	357

istics of the stop. Based on the opinion of the observer who rode with the driver on each test, the most uncomfortable stops, or those most likely to throw an occupant against the windshield or dashboard, were the stops made from the lowest speed of 20 miles per hour. At high speeds, the brakes are evidently not capable of overcoming the angular momentum of the wheels and other revolving parts of the vehicle at a fast enough rate to cause a sudden and great change in the speed of a vehicle.

Using data for the tests in which the drivers applied the maximum brake pressures they could develop, it was found that no wheels locked in 42 percent of the tests, all wheels locked in 30 percent, and one or two wheels—generally the rear wheels—locked in 28 percent of the tests. Wheels were more apt to lock at the lower speeds than at the higher speeds. At speeds of 20 miles per hour, all four wheels locked in 35 percent of the tests, whereas at speeds exceeding 60 miles per hour the corresponding figure was only 19 percent. The common assumption that any passenger car with brakes in good condition is capable of locking all four wheels may be questioned. It may hold true for certain drivers on all types of surfaces or for all drivers on certain surfaces, but not for all drivers on all surfaces. The concrete surface on the taxiway had a higher coefficient of friction than the surface of any highway on which tests were made (as described later) in connection with this investigation. A dangerous condition would exist, however, if manufacturers provided brakes that would grab or lock the wheels too easily.

Square-of-Speed Rule in Error

The braking distance does not vary as the square of the speed. For example, the average stop from 30 miles per hour was made in 40 feet. With the braking distance varying as the square of the speed, a stop from 90 miles per hour would require only 360 feet, whereas the average stop from 90 miles per hour actually required 580 feet. Not one of the stops from 90 miles per hour was made in 360 feet. The shortest distance was 490 feet.

There are several reasons why the braking distances do not vary as the square of the speed. The effects of brake fade have already been discussed. Likewise it has been shown that full deceleration does not start immediately when the brake pedal is touched. It takes some time to depress the pedal as far as it will go and some additional time for the brake fluid to expand the brake shoes through the wheel cylinders. Other factors also affect the brake distance. While it is true that the kinetic energy of the vehicle in the direction it is traveling varies as the square of the speed, the rate at which brakes can absorb this energy and the additional angular kinetic energy in the wheels and other rotating parts is apparently limited.

The relation between speed and braking distance as obtained by these tests is shown in figure 12, in which two curves are presented. The lower curve shows average stopping distances, and the upper curve shows the 85-percentile stopping distances. The equation for the lower curve is:

$$D = 0.00101V^{2.92} + 0.82V$$

where D is the average stopping distance in feet and V is the speed in miles per hour.

The constants in this equation were obtained by assuming that the equation should have the form $D = aV^b + cV$. They were determined so as to minimize the sum of the squared deviations of the plotted points from the curve. The second term on the right-hand side of the equation represents the distance traveled during actions taking a constant length of time, such as the time to depress the brake pedal and the time required for the brake cylinders to expand the brake shoes. This equation fits the data better than any other type of equation that was investigated and far better than one based on the assumption that braking distances vary as the square of the speed.

The 85-percentile curve in figure 12 shows the distance within which 85 stops out of 100 can be made and not the distance within which 85 percent of the vehicles can always stop. It has been based on observed data up to a speed of 40 miles per hour. Above 40 miles per hour there were so few tests that the 85-percentile curve has been based on sound statistical procedures assuming that the second term, $0.82V$, which consists largely of brake-application distance, and the exponent in the first term would be the same as for the average curve. It was also known that most of the vehicles with poor brake performance at the lower speeds were not tested at the higher speeds. In fact, only 3 of the 11 vehicles that required more than the 85-percentile distance to stop from a speed of 20 miles per hour participated in the tests at speeds above 40 miles per hour. Also, an analysis of the stopping

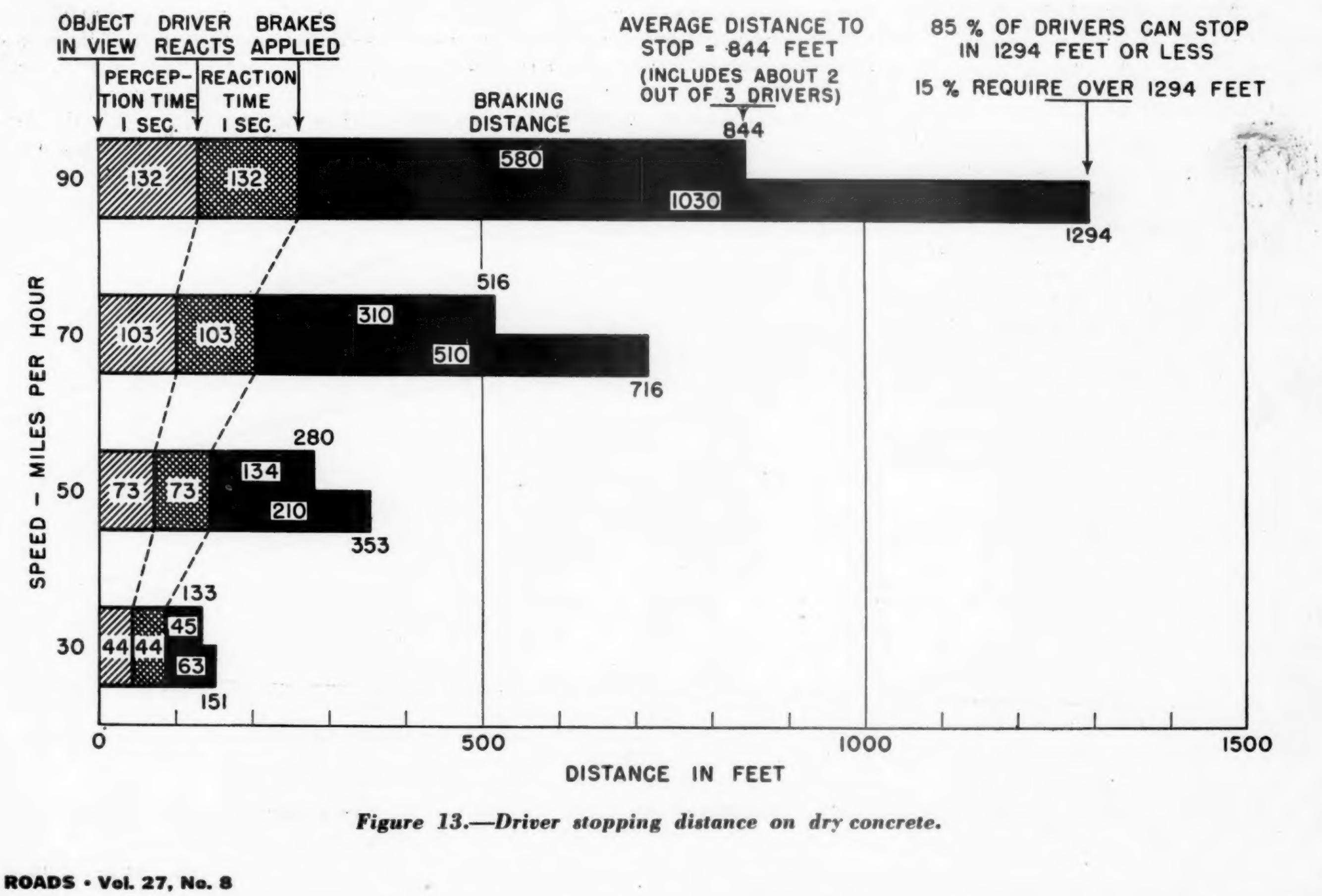

Figure 13.—Driver stopping distance on dry concrete.

distances of the vehicles that did participate at the high speeds shows a marked tendency for those having the longer stopping distances at the lower speeds also to have the longer stopping distances at higher speeds. The portion of the 85-percentile curve above 40 miles per hour therefore represents the best estimate that can be made on a basis of these tests even though few of the points shown in figure 12 represent stopping distances longer than the 85-percentile values.

Equations that are somewhat easier to solve and give approximately the same results as the equations shown in figure 12 are as follows, for the average and 85-percentile curves, respectively, with D and V being in the same units as for the exact equations:

$$D = 0.00069V^3 + V$$
$$D = 0.00126V^3 + V$$

An appreciation of the effect of speed on the distance required to stop a passenger car may be obtained from figure 13. The 1-second perception time and the 1 second for reaction time are certainly minimum values for drivers under actual operating conditions except possibly under congested traffic conditions when the time required by a driver to perceive the illumination of the stop light on a preceding vehicle might possibly be somewhat less than 1 second. Considering the conditions under which these data were obtained, the driver stopping distances shown in figure 13 can be considered as absolute minimums for use in determining standards of design for safe highways. If any safety factor is applied, longer driver stopping distances must be used. For certain elements of design the average values might be applicable, but safe conditions generally will not be attained unless driver stopping distances at least as high as the 85-percentile values are used.

Coefficients of Friction

Some surfaces when dry do not have as high a coefficient of friction as a concrete surface, and no road surface included in these tests had as high a coefficient of friction when wet as a dry concrete surface. The 85-percentile values for the driver stopping distance as shown in figure 13 are applicable to all road surfaces, however, where the coefficient of friction that can be developed between the tires and the road surface is equal to or greater than the lowest equivalent coefficient of friction utilized in these tests by the vehicles in making 85 percent of the stops.

The average coefficients of friction utilized by the vehicles over their entire braking distances on the dry concrete surface are shown in figure 14. These averages were calculated by use of the equation $d = V^2 \div 30f$ where d is the braking distance in feet, V is the initial speed in miles per hour, and f is the average coefficient of friction developed between the tires and the road surface over the entire braking distance. Since the braking distance includes the distance traveled during the brake application time, the average utilized coefficient of friction increases as the speed increases from 20 to 30 miles per hour because the brake application distance becomes an increasingly smaller portion of the total braking distance.

While making stops from 20 miles per hour, there was at least one vehicle that utilized an average coefficient of friction of 0.88 over its braking distance. The average coefficient of friction utilized by the average vehicle was 0.65, and the vehicles that required the 85-percentile braking distance utilized an average coefficient of 0.48. Likewise, the corresponding coefficients of friction that were utilized for the stops from 90 miles per hour were 0.57, 0.46, and 0.26 respectively. The maximum friction coefficient during any one stop is always greater than the average for the entire braking distance. For example, during the stop from 30 miles per hour as shown in figure 10, the vehicle could have stopped in the 49 feet if a coefficient of friction of 0.61 had been utilized over the entire distance, calculated on the same basis as the curves of figure 14. Actually, however, the maximum coefficient of friction developed during this same stop was 0.72 since the maximum deceleration rate was 23.2 feet per second per second as shown in figure 9. It is evident, therefore, that coefficients of friction greater than the average values shown in figure 14 were developed between the tires and the road surface during the braking-distance tests.

In an effort to determine the actual coefficient of friction of the taxiway surface and to compare this value with the coefficients utilized by the vehicles in braking, equipment was developed to measure friction coefficients of road surfaces. This equipment and other tests for which it was used will be described after the results of friction tests on the taxiway surface have been discussed.

Tires and Friction Factors

Coefficients of friction between the taxiway surface and one particular set of tires under dry conditions were found to be as shown in the column of table 3 headed "first set of tires." It was evident that if these were the correct coefficients of friction for the taxiway surface, it would have been impossible to make many of the stops within the distances recorded during the brake tests. Average coefficients of friction at least as high as those shown in figure 14 were necessary to stop the vehicles within the recorded distances, excluding the braking effect of air resistance.

The only explanation for the wide discrepancy seemed to be that the tires on the vehicle used to measure the friction coefficients were not as resistant to skidding as the tires on some of the vehicles used for the brake tests. This was confirmed by interchanging the tires on the friction-test vehicle with the tires on one of the vehicles for which short stopping distances were recorded. The coefficients of friction for the surface on the taxiway were then found to be from 23 to 33 percent higher than with the original tires, as shown in the last column of table 3. The coefficients were then of sufficient magnitude to account for the braking distances recorded during the high-speed tests in which high coefficients of friction were utilized.

Similar differences in friction coefficients between the two sets of tires were found for a bituminous surface, the results for which are included in table 3. It was surprising to find that the tires made such a great difference, especially since both sets of tires had the same tread pattern and were fabricated from the same rubber compound (based on the manufacturer's records of the serial numbers).

The hardness to which the rubber had been cured appeared to be the only measur-

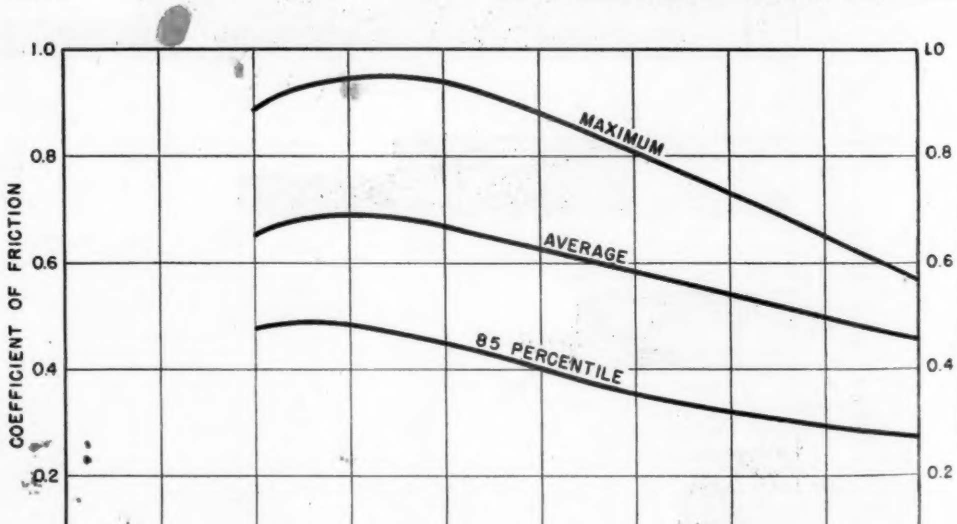

Figure 14.—Average utilized friction coefficient while braking.

Table 3. — Effect of tires on friction coefficients

Type of test	Coefficient of friction	
	First set of tires	Second set of tires
Concrete surface on taxiway:		
Slide impending from stopped position	0.77	0.95
Slide impending with wheels turning at slow speed	.69	.92
Sliding at slow speed	.67	.86
Slide impending at 25 miles per hour	.57	....
Sliding at 25 miles per hour	.46	....
Bituminous concrete surface:		
Slide impending at slow speed	.56	.70
Sliding at slow speed	.53	.68

able difference between the physical characteristics of the two sets of tires. Had an attempt been made to find the tires that would result in the highest and lowest friction factors, the difference might well have been much greater than the difference between the two sets that were used.

A few tests were also made on a third set of tires by towing a light pickup truck with a large 10-wheeled wrecker. The resulting coefficients of friction were 53 percent higher than for the first set of tires and 25 percent higher than for the second set of tires. The third set of tires was one size larger and had a different tread pattern than the other two sets of tires. The rubber used in their fabrication may also have been of a different compound and cured to a different hardness than either of the other two sets of tires.

In view of these results, further study should be made to obtain conclusive answers to a number of questions directly related to highway safety. How many drivers realize when they buy a new set of tires that their stopping distances in emergencies may be 30 percent greater with one set of tires than with another set? For safety reasons, would it be equally desirable to reduce the nonuniformity in tires as the nonuniformity in the texture of road surfaces to improve coefficients of friction between tires and road surfaces? Also, in brake-performance tests, to what extent are coefficients of friction between the road surface and the particular set of tires being measured rather than brake performance?

Further Friction Studies

The studies on the taxiway surface were for the purpose of obtaining some idea of the relation between friction coefficients and stopping distances. While the equipment for measuring friction coefficients was available, it appeared desirable to extend the studies to other surfaces, principally to obtain a better idea of the problems involved in programming a comprehensive study of the relation between friction coefficients and braking distances.

To measure friction coefficients of road surfaces, a four-wheeled vehicle was towed with a cable and the towing force measured with a resistance strain-gage dynamometer. This method was selected as the most suitable one available after reviewing other methods reported for previous investigations.

The towed vehicle used for these tests was an Army jeep with a four-wheel drive, new brakes, and passenger-car tires. Two hydraulic shock absorbers were mounted on the front bumper in such a manner that they served as the front support for the tension-bar dynamometer used in conjunction with an electronic strain recorder. The shock absorbers also served to dampen vibrations and variations in the towing force imparted to the dynamometer by the cable when on rough surfaces. The other end of the dynamometer was fastened to the bumper of the jeep with a connection that permitted the same flexibility as a universal joint. The towing force was kept parallel with the road surface at all times by a 25-foot steel cable attached to the differential of the tow truck at the same height as the mounting on the bumper of the jeep. Figure 15 shows the method of connecting the electric dynamometer to the towed vehicle, and figure 16 shows a typical dynamometer recording.

A resistance strain-gage dynamometer and an electronic recorder furnished by the Naval Gun Factory were used throughout these tests. Gages were placed on the four sides of the half-inch-square aluminum bar with a temperature control element. The unit was waterproofed to permit operation in any type of weather. The SR-4 strain recorder was fastened to shock mountings in the tow truck and connected with the dynamometer bar by insulated wires fastened to

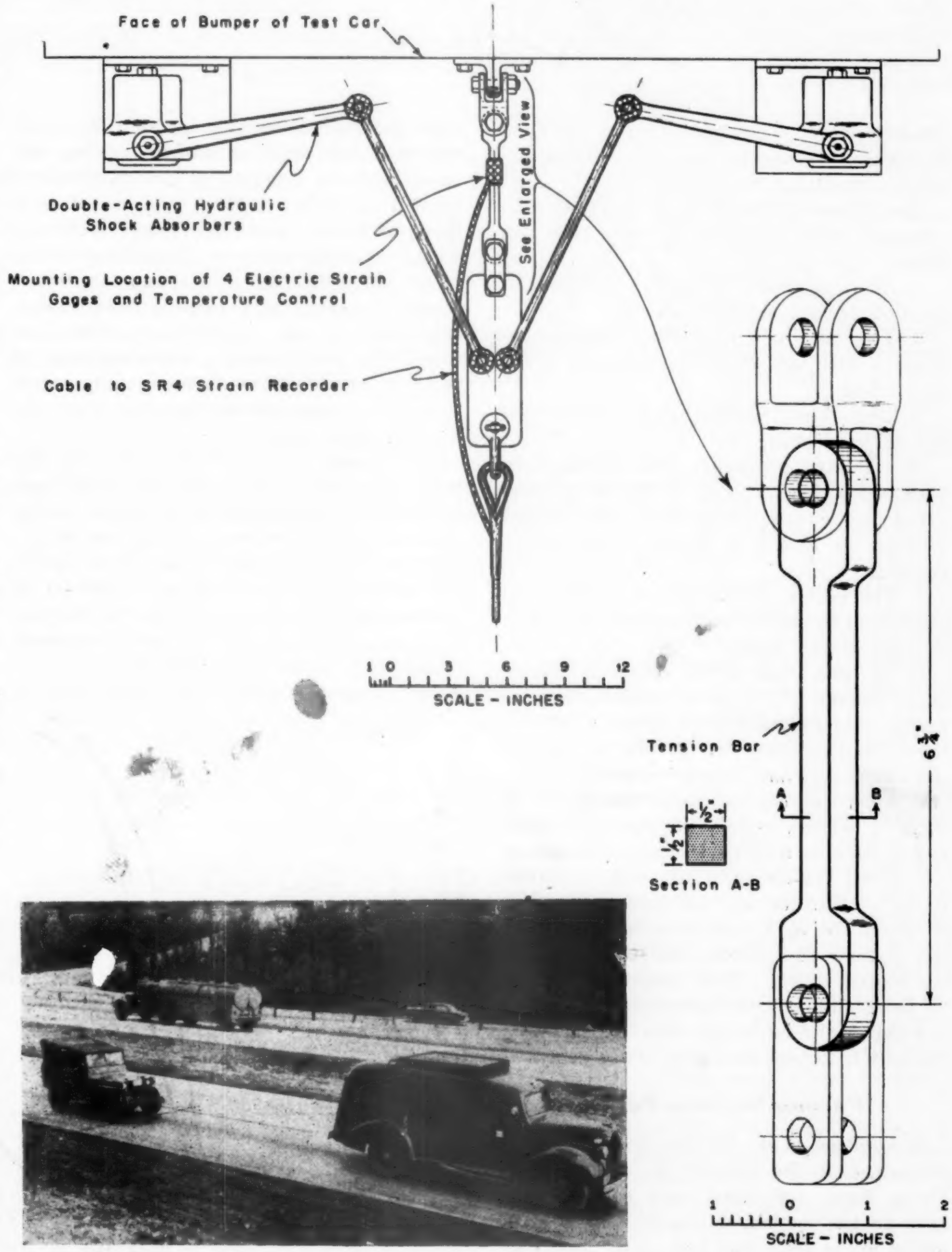

Figure 15.—Method of connecting electric tension dynamometer to towed vehicle. Inset—towing jeep with panel truck.

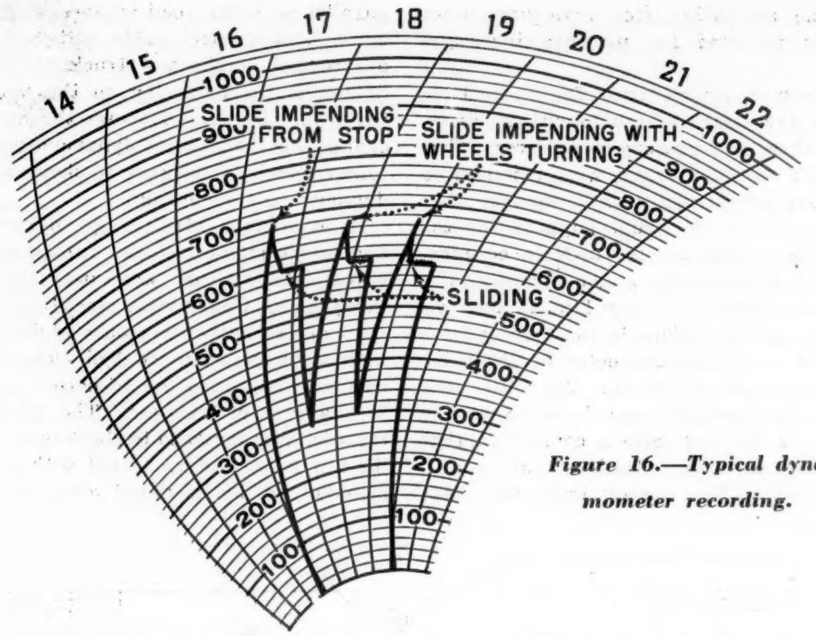

Figure 16.—Typical dynamometer recording.

the tow cable. The calibration of the dynamometer was checked before starting and after completing the friction tests.

The following three types of friction coefficients were measured at each test location:

1. The maximum starting coefficient of friction from a stopped position.
2. The maximum impending coefficient of friction with the wheels turning and a skid impending.
3. The sliding coefficient of friction with the wheels locked.

A standard procedure was followed for each test which involved the following steps (these were repeated at least once at each location to obtain a check on the initial readings):

1. The brakes of the jeep were fully applied with the motor stopped and the transmission in low gear.
2. The tow truck moved ahead in low-low gear, engaging the clutch slowly until the towed vehicle started to slide. The tow truck then continued at a slow speed until the entire test had been completed.
3. After the jeep had moved about 10 feet with its wheels locked, the operator disengaged the clutch and released the brakes on the towed vehicle until the wheels started to turn. He then applied the brakes slowly to obtain the maximum braking force without locking the wheels and then released the clutch slowly. This caused all wheels to start sliding simultaneously.
4. After sliding ahead about 10 feet with the wheels locked, operation 3 was repeated.

Various Surfaces Tested

In addition to the friction tests that were conducted on the taxiway at Andrews Air Force Base, 108 tests were conducted in 1950 on the section of concrete on U S 40 where brake tests had previously been conducted on vehicles selected at random. Friction tests were also conducted at 25 locations selected at random around Washington, D. C., to obtain some idea of the various conditions encountered by drivers in the normal operation of their vehicles. One of these locations was on Memorial Bridge where rear-end collisions frequently caused long delays to traffic crossing the bridge during morning and evening rush hours, especially on days when the surface was wet. The measurements were repeated at 11 of the test locations during a rainy period or between intermittent showers while the surfaces were wet.

The results of these tests show that the taxiway on which the brake tests from high speeds were conducted had a higher coefficient of friction when dry than any of the road surfaces on which tests were made. The taxiway had an average coefficient of friction 5 percent higher than the surface in Maryland where the random brake-performance tests were conducted.

The range in friction coefficients on a 1-mile section of highway on and in the vicinity of the half-mile section where the random brake-performance tests were conducted is shown in figure 17. These were obtained with the set of tires that resulted in the lower coefficients, but the range shown in figure 17 is nevertheless significant.

With a range of more than 10 percent in the coefficient of friction of the surface, depending on the exact location of the test, and a possible variation of at least 30 percent due to the tires, it is evident that these variations had some effect on the results of the brake-performance tests. Was brake performance or the coefficient of friction between the road surface and the tires being measured? It is evident that the results for one must be considered in combination with the other.

The relative coefficients of friction at low speeds for seven different types of surfaces when dry and wet are shown in figure 18. The term "relative" is used because the values are based on one particular set of tires. With other sets of tires the values might be considerably higher or lower and there is not any positive assurance that the relative magnitude of the coefficients between different surfaces would be the same as shown in figure 18. In fact, the results of a limited number of tests on different surfaces with different tires indicate the possibility that one surface with a relatively high coefficient of friction as compared with other surfaces when measured with one set of tires may have relatively a much lower coefficient of friction when the measurements are made with another set of tires. It can be hoped, however, that the data shown in figure 18 do represent about average conditions on a relative basis. They should not be used without this qualification and without the additional qualification that construction methods and other factors can result in a wide variation in friction coefficients even for the same general type of surface.

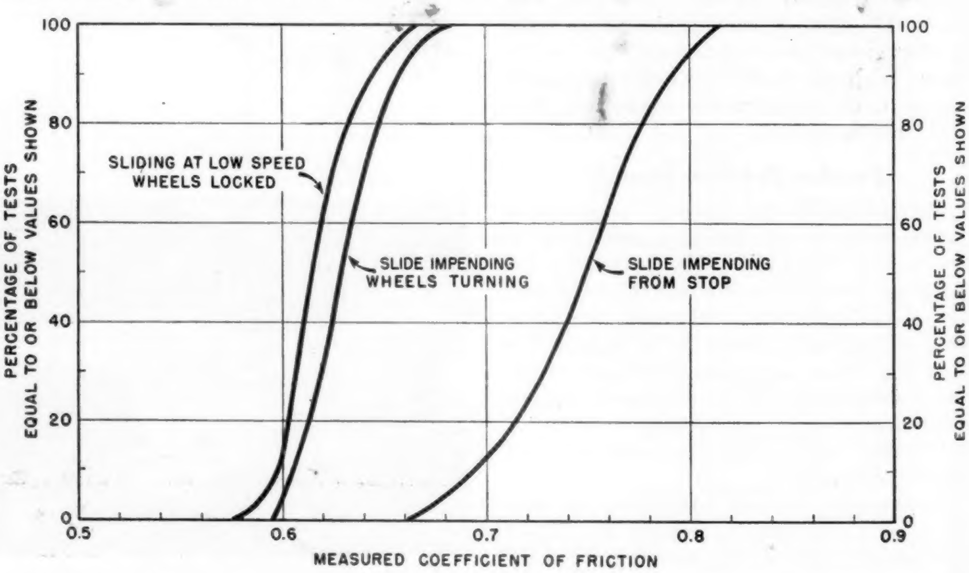

Figure 17.—Range in coefficients of friction on 1-mile section of concrete pavement where random brake-performance tests were conducted.

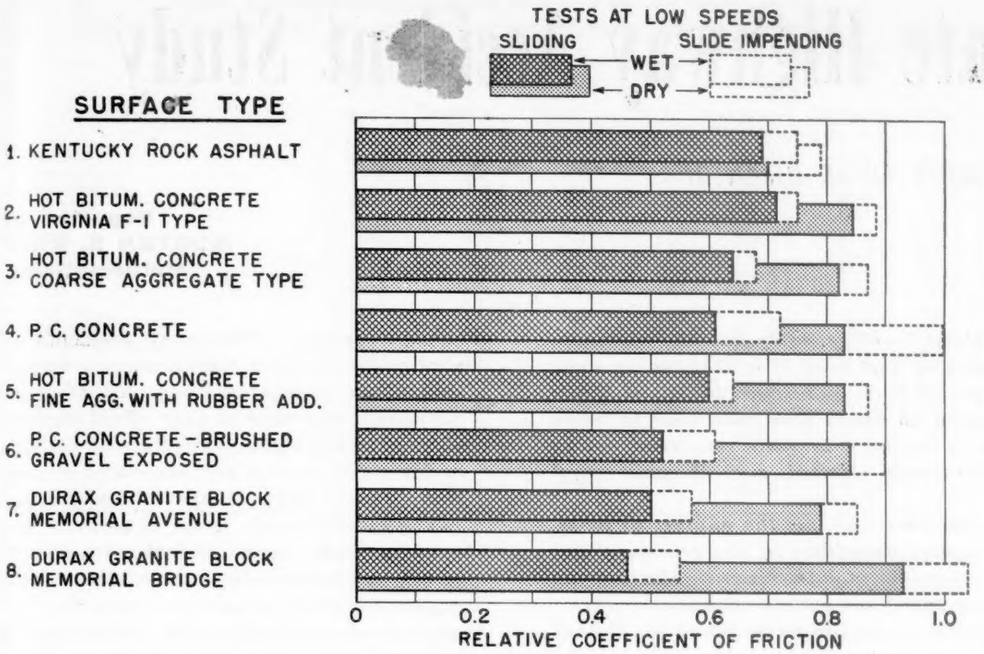

Figure 18.—Relative coefficients of friction (impending coefficient for portland cement concrete used as unity).

Figure 18 shows that the impending coefficients of friction immediately before the wheels start sliding are higher for all types of surfaces, both dry and wet, than the coefficients after the wheels start to slide. Also, all surfaces when wet have lower friction coefficients than when dry.

Interesting Comparisons Made

A most interesting comparison exists between the Durax granite block surface on Memorial Bridge, where heavy traffic volumes had worn the peaks on the blocks smooth, and the same type of surface on Memorial Avenue, where traffic volumes since construction had been much lower than on the bridge. When dry, the worn surface on the bridge had a higher coefficient of friction than any of the road surfaces, but when wet it had the lowest coefficient of friction. Both the impending and sliding values for the wet condition were only about 50 percent of the values for the dry condition. This was undoubtedly an important contributing factor to the large number of rear-end collisions and at least two head-on collisions that occurred while the surface was wet.

When Memorial Bridge was resurfaced with a rock asphalt mixture, the coefficient of friction for the dry condition was reduced about 25 percent. During the more critical condition, however, when the surface was wet, resurfacing the bridge increased the impending coefficient of friction 36 percent and the sliding coefficient of friction 53 percent.

It is also interesting to note that all of the surface materials except Kentucky rock asphalt and the granite blocks have about the same coefficients when dry and a wide range in the coefficients when wet. At one intersection, the intersection area and the approaches for a short distance from the intersection had been resurfaced with a hot bituminous concrete containing a fine aggregate and a rubber addition as an experiment. The resurfaced areas (type 5 in figure 18) actually had a lower friction coefficient than the sections on the approaches that had not been resurfaced (type 2).

The effect of temperature on friction coefficients was studied at 12 locations by conducting tests while the air temperature was 36°F. on one day and repeating the tests on another day when the temperature was 53°. It was cloudy on both days, and there had been a change in air temperature of only 2° during the 24 hours prior to the tests.

The difference in temperature apparently had no effect at five locations with concrete surfaces. At three locations, the coefficients of friction were slightly higher, and at two locations they were slightly lower at 53° than at 36°. At the seven locations with various types of bituminous surfaces, the friction coefficients were consistently higher at the lower temperature than at the higher temperature. The average difference was 10 percent for the impending coefficients of friction with the wheels turning and 8 percent with the wheels sliding in a locked position.

Need for Further Research

The tests to determine braking distances of vehicles from high speeds and the tests of friction coefficients may be regarded as pilot studies pointing to the need for far more extensive studies that should involve the cooperation of the automotive and tire industries. The brake-performance tests may seem to be of limited number but, as far as is known, they have not been made elsewhere in larger numbers.

The tests were of sufficient scope to throw serious doubt on some of the beliefs and opinions accepted in the past and to suggest the need for research broad enough in scope to give conclusive answers to the following questions:

1. What is the braking force or deceleration rate that drivers should be expected to attain on a dry surface? The tests showed that drivers of passenger cars with brakes in good condition were not always capable of obtaining a braking force sufficient to lock the wheels on dry surfaces with high coefficients of friction, especially at high speeds.

2. Are not distances within which most vehicles can be stopped from high speeds most of the time on surfaces with high coefficients of friction considerably longer than those generally accepted to be correct in the past? In these limited tests vehicle braking distances on dry surfaces increased with increased speed at a rate greater than the square of the speed.

3. What causes the large variation in the braking distances of different vehicles and of the same vehicle during successive stops? During these tests there was a wide variation in the braking action of the same vehicle and in the braking action of different vehicles, especially at high speeds.

4. To what extent do brakes and road conditions cause vehicles to dive to one side and the drivers to lose control of their vehicles? In these tests deceleration rates attained by vehicles in making stops from high speeds were not uncomfortable unless the wheels locked or the driver was unable to control the path of the vehicle due to improperly adjusted brakes or nonuniformity of the road surface.

5. What is the exact extent to which brake fade and other factors affect stopping distances of vehicles under normal operating conditions? In these tests brake fade appeared to be one of the most serious deficiencies of the cars tested.

The results of the friction coefficient measurements are useful principally as a pilot study to illustrate the necessary magnitude of any investigation designed to obtain exact information on the interrelation of stopping distances and friction coefficients between tires and road surfaces. Much is being done by highway departments to improve the nonskid qualities of roadway surfaces and to eliminate types that are exceedingly slippery when wet. Is it not also necessary to consider the variation in tires concurrently with the road surfaces to improve friction coefficients? Improvement in operating safety can be expected from continued attention to better road surface design and from continued improvement in the construction of tires. Would it not be advisable to establish minimum standards for both road surfaces and tires to avoid having drivers confronted with unexpectedly hazardous friction factors on both wet and dry surfaces?

The Interstate Highway Accident Study

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by
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While highway designers often talk about building safety into the highways, there have been few comprehensive studies of just what it is that makes some roads safer than others. In the study reported here, more than 16,000 accidents on approximately 5,000 miles of highway in 15 States have been used to relate the accident rates on different sections of highway to their respective design features and traffic characteristics. The roads selected are all main rural highways.

The most significant factors affecting accident rates are the number of lanes, the volume of traffic, the degree of curvature, the widths of the pavement and the shoulders, and the percentage of cross traffic at intersections. Most of the effects are in the expected directions, but there are certain exceptions.

At high traffic volumes, the lowest accident rates are to be found on divided roads with controlled access, while the highest rates occur on three-lane roads. On most types of highway sections the accident rate becomes higher with increasing traffic volume, except for a slight reversal due to congestion at extremely high volumes. However, there is a different pattern for two-lane curves and intersections, where the accident rate declines as the volume increases.

Sharp curves have higher accident rates than flat curves on roads carrying the same amount of traffic. Wide pavements and shoulders are conducive to safety on two-lane curves, though they do not appear to have any value on two-lane tangents. Bridges are greatly helped by having the bridge roadway several feet wider than the approach pavement.

At intersections, the percentage of the total traffic which is on the minor road is extremely important. Intersections where the cross traffic is between 10 and 20 percent of the total have more than twice the accident rate of intersections having less than 10 percent cross traffic. Also, three-way intersections are considerably safer than four-way crossings.

A number of roadway features failed to show any consistent relation to accident rates. These include grade, the frequencies of curves and sight restrictions, and the percentages of commercial and night traffic.

A POPULAR VIEW has it that every accident results from some "principal cause," like speeding or driving on the wrong side of the road. The way to prevent accidents, according to this view, is to stop drivers from doing the things that stand highest on the list of principal causes.

The problem is not so simple. Every accident has many causes, if we consider a cause to be any remediable condition whose correction would have prevented the accident. For example, suppose two cars driven at high speed have a head-on collision at night on a two-lane road. The speed of the vehicles is obviously one cause of the collision. Other causes may be the use of blinding headlights, the absence of lighting on the highway, inadequate pavement width, and the fact that the road carries two-way instead of one-way traffic. Any of these may have contributed equally with speed to the accident, but the chances are that speeding will get most of the blame.

It is desirable to examine all the causes of an accident instead of concentrating on a single cause, since a wide variety of corrective measures may be suggested by a broad approach. Classifying accidents according to all their circumstances, regardless of how unimportant any particular circumstance may seem at first, offers the hope of discovering significant relations between accident frequencies and associated circumstances which might otherwise escape notice.

The present study is an attempt to find out how rural traffic accident rates are affected by various physical features of the highway and by certain use characteristics such as average daily traffic, percentage of commercial vehicles, and the like. These are by no means the only causes of accidents. But if it should turn out, for instance, that roads with flat curves are appreciably safer than roads with sharp curves, then it would be possible to predict with some assurance the accident savings that would result from building flatter curves into the highways. Accidents have many causes, and an effective accident-reduction program ought to use the full range of remedies. This study is intended to throw light on those remedies which are in the domain of the highway designer.

Summary of Findings

The most significant factors affecting accident rates, as discovered in this study, are

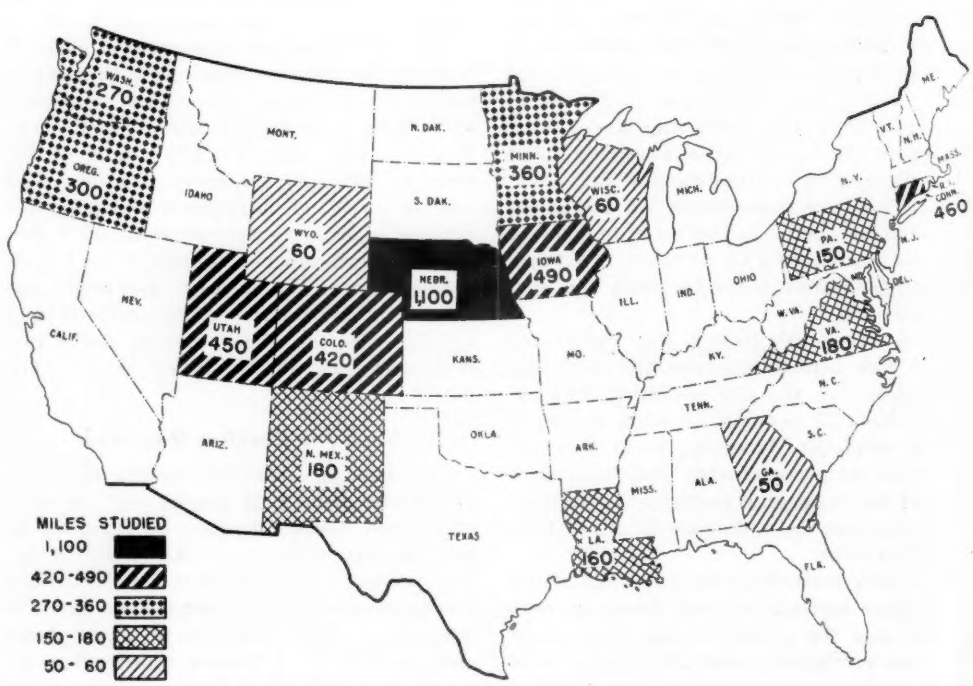

Figure 1.—Mileage of study routes.

Table 1.—Length and amount of travel on the study routes

State	Study year	Approximate length, in miles			Travel in study year, in million vehicle-miles		
		Tangents	Curves	Total	Tangents	Curves	Total
Colorado	1941	350	70	420	380.4	77.3	457.7
Connecticut	1941	300	150	450	639.1	249.0	888.1
Connecticut	1946	310	150	460	508.6	200.0	708.6
Georgia	1940	40	10	50	40.5	13.7	54.2
Iowa	1941	430	60	490	303.2	41.9	345.1
Louisiana	1941	130	30	160	175.0	30.9	205.9
Minnesota	1946	290	70	360	180.4	48.0	228.4
Nebraska	1941	1,000	100	1,100	547.5	57.3	604.8
New Mexico	1941	160	20	180	64.9	5.0	69.9
Oregon	1941	210	90	300	220.9	58.0	278.9
Pennsylvania	1941	100	50	150	133.2	74.0	207.2
Utah	1941	370	80	450	238.9	46.0	284.9
Virginia	1941	150	30	180	275.4	54.9	330.3
Washington	1941	200	70	270	469.1	135.8	604.9
Wisconsin	1941	50	10	60	70.3	11.0	81.3
Wyoming	1941	50	10	60	22.4	4.4	26.8
Total		4,140	1,000	5,140	4,269.8	1,107.2	5,377.0

traffic volume, degree of curvature, pavement and shoulder width on curves, percentage of cross traffic at intersections, and the width of bridge roadways, both absolutely and in relation to their approach pavements. In most cases the effects are in the expected directions, but there are certain exceptions.

Volume of traffic has a strong effect on the accident rate on nearly all types of highway sections. In general—except for curves and intersections on two-lane roads—the accident rate becomes higher as the volume is increased. There is often a slight reversal of this trend at very high volumes, presumably because extreme congestion inhibits the drivers' ability to make passing maneuvers.

At curves and intersections on two-lane roads the trend goes the other way. Here the accident rates become lower with increased traffic volume. This effect has been well substantiated, but the reason for it remains a matter of speculation. A plausible theory is that the two-lane curves and intersections present conditions which most drivers recognize as hazardous, particularly when there is a considerable amount of traffic. Accordingly, the driver pays enough extra attention when these facilities are busy to more than compensate for the added potential danger.

Sharp curves have higher rates than flat curves. The volume effect described above is more pronounced on sharp curves than on flat ones.

Wide pavements and shoulders help to reduce the accident rates on two-lane curves. This is in contrast to the two-lane tangents, where no particular effect could be traced to the width of the pavement or the shoulders.

The percentage of cross traffic at an intersection has a tremendous effect on its accident rate. It takes only about 15 percent cross traffic to make an intersection more than twice as hazardous as when the cross traffic is well below 10 percent.

Another important intersection characteristic is the number of approaches. Three-way intersections (T and Y) have markedly lower accident rates than four-way crossings. This is not necessarily an argument for staggering all crossings, however, as the increased number of intersections and the additional turning and weaving might easily nullify the apparent advantage.

Wide roadways are desirable at two-lane bridges and underpasses, and they should be several feet wider than the approach pavements. Of the two types of structures, underpasses are considerably more hazardous.

A number of roadway features did not appear to have any consistent effect on the accident rates. These include grade, pavement and shoulder widths, frequency of curves, frequency of sight restrictions, and the percentages of commercial and night traffic.

Technique of Study

To make it possible to study a large number of different highway features, the roads included in the study were divided into short homogeneous sections. Each of these sections was substantially uniform in grade, pavement width, shoulder width, degree of curvature, traffic volume, etc. Any place where a change occurred in any of these characteristics was made a dividing line between one section and another. Intersections and structures were also regarded as sections, because of their special characteristics

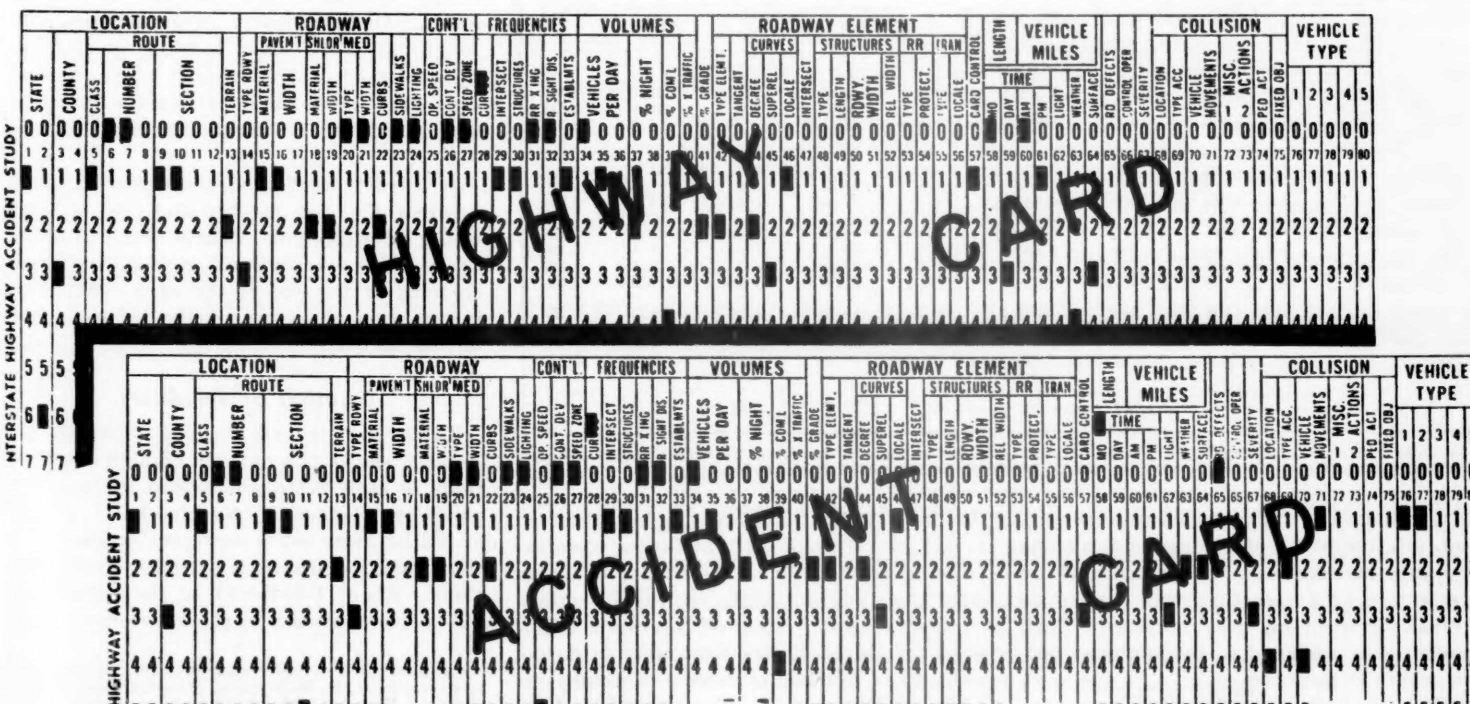

Figure 2.—A highway punch card and an accident punch card on the same highway section.

Table 2.—Number of highway sections, by type of roadway element

State	Tangents	Curves	Inter-sections	Structures	Railroad crossings	Other	Total
Colorado	1,239	571	604	207	13	2	2,636
Connecticut (1941)	1,634	1,356	1,027	311	9	106	4,443
Connecticut (1946)	1,628	1,317	1,020	329	9	106	4,409
Georgia	160	84	16	6	0	0	266
Iowa	1,434	493	686	293	13	0	2,919
Louisiana	333	148	108	64	19	11	683
Minnesota	1,402	533	612	54	13	2	2,616
Nebraska	2,683	559	1,107	298	34	0	4,681
New Mexico	416	77	16	58	0	0	567
Oregon	626	771	322	122	9	0	1,850
Pennsylvania	338	239	12	203	0	11	803
Utah	896	530	276	51	18	27	1,798
Virginia	832	240	263	92	2	14	1,443
Washington	1,093	610	537	83	17	0	2,340
Wisconsin	232	69	91	16	2	2	412
Wyoming	132	58	30	4	1	0	225
Total	15,078	7,655	6,727	2,191	159	281	32,091

Table 3.—Number of accidents, by type of roadway element

State	Tangents	Curves	Inter-sections	Structures	Railroad crossings	Other	Total
Colorado	676	130	225	80	0	0	1,111
Connecticut (1941)	1,374	560	426	11	0	16	2,387
Connecticut (1946)	1,223	439	287	81	1	67	2,098
Georgia	64	32	17	8	0	0	121
Iowa	461	100	98	20	1	0	680
Louisiana	531	39	107	14	2	0	693
Minnesota	254	46	170	13	5	0	488
Nebraska	638	77	121	29	6	0	871
New Mexico	89	10	9	1	0	0	109
Oregon	926	106	366	118	0	0	1,516
Pennsylvania	265	140	22	19	0	55	501
Utah	447	120	116	16	7	0	706
Virginia	648	164	250	51	0	2	1,115
Washington	1,962	509	1,119	86	17	1	3,694
Wisconsin	197	7	68	2	0	0	274
Wyoming	45	4	8	0	0	0	57
Total	9,800	2,483	3,409	549	39	141	16,421

Table 4.—Number of accidents, by severity, and ratios and adjustment factors

State	Number of accidents				Ratio of—			Adjust-ment factor [1]
	Fatal	Injury	Other	Total	Total to fatal	Total to fatal-plus-injury	Injury to fatal	
Colorado	46	382	683	1,111	24.2	2.60	8.3	2.07
Connecticut (1941)	51	1,018	1,318	2,387	[2] 46.8	2.23	20.0	1.07
Connecticut (1946)	42	758	1,298	2,098	[2] 50.0	2.62	18.0	1.00
Georgia	8	47	66	121	15.1	2.20	5.9	3.31
Iowa	30	274	376	680	22.7	2.24	9.1	2.20
Louisiana	22	260	411	693	[2] 31.5	2.46	11.8	1.59
Minnesota	10	178	300	488	[2] 48.8	2.60	17.8	1.02
Nebraska	46	406	419	871	18.9	1.93	8.8	2.65
New Mexico	11	59	39	109	9.9	1.56	5.4	5.05
Oregon	39	288	1,189	1,516	[2] 38.9	4.64	7.4	1.29
Pennsylvania	20	193	288	501	[2] 25.0	2.35	9.7	2.00
Utah	33	283	390	706	21.4	2.23	8.6	2.34
Virginia	102	434	579	1,115	10.9	2.24	4.3	4.59
Washington	84	975	2,635	3,694	[2] 44.0	3.49	11.6	1.14
Wisconsin	9	89	176	274	[2] 30.4	2.80	9.9	1.64
Wyoming	4	27	26	57	14.2	1.84	6.8	3.52
Total	557	5,671	10,193	16,421	29.5	2.64	10.2	....

[1] 50 divided by the total-to-fatal ratio. These adjustment factors are used in computing the type 1 accident rates.
[2] Type 2 accident rates are computed only for those States with total-to-fatal ratios of at least 25.

(e.g., volume of traffic on the intersecting road, relative width of bridge roadway and adjoining pavement). The presence of an intersection or a structure was treated as a break between highway sections. Each accident was assigned to the highway section where it occurred.

The basic techniques for the study were devised by the National Safety Council in cooperation with the Bureau of Public Roads in 1945, following a pilot study on U S 1 in Virginia. All 48 States were invited to participate in the study, using as data the 1941 accidents on rural sections of the National System of Interstate Highways. Only 15 States were able to do so, although a number of others expressed interest in the project. The year 1941 was selected as the most recent in which driving conditions were "normal." A few of the States used other years (see table 1), and some main rural highways were used which are not part of the National System of Interstate Highways. The chief obstacles to wider participation by the State highway departments were insufficient manpower to prepare the strip maps and the coding sheets, inability to locate accidents accurately, and incomplete accident reporting, in the sense that too many accidents went unreported. Two brief reports of preliminary findings have already been published.[1]

The participating States are given in figure 1 and table 1. For each State the table gives the year for which accident records were submitted, the number of miles of roadway included in the study, and the total amount of travel on those roads during the study year. (In addition to 1941 data, Pennsylvania submitted cards for accidents in the first 5 months of 1942, but these have not been used in any of the analyses covered in this report.)

The data were recorded on tabulating cards, the coding procedure calling for two sets. One set, called highway cards, contains a card for each highway section. The second set, called accident cards, contains a card for each accident. In figure 2, the upper picture is a typical highway card; below it is an accident card representing an accident on the same highway section. The first 56 columns, which identify and describe the highway section, are identical on the two cards. This makes it possible to classify accidents according to various highway features without having to refer to the highway cards.

The punch in column 57 indicates which type of card it is. The remaining columns serve different purposes on the two types of cards. On a highway card they give the length of the section and the annual vehicle-mileage of travel on it. On an accident card these columns contain information about the circumstances of the accident.

A third type of card, the summary card, has recently been punched and used in some of the later analyses. There is one of these cards for each highway section, with the columns at the end of the card containing information about the number of accidents on the section.

Number of Accidents

Tables 2–4 indicate the size of the study. Table 2 shows the number of highway sections in each State, subdivided by types of roadway elements. Nearly half of the 32,091 highway cards represent tangent sections, and one-fourth represent curve sections. About two-thirds of the remainder (21 percent of the total) represent inter-

[1] *A plan for relating traffic accidents to highway elements*, by C. F. McCormack. American Association of State Highway Officials Convention Group Meetings, 1944, pp. 117-119. *The relation of highway design to traffic accident experience*, by D. M. Baldwin. American Association of State Highway Officials Convention Group Meetings, 1946, pp. 103-109.

sections, and the rest are for structures, railroad crossings, toll stations, and transitions of various kinds.

Table 3 shows similar information regarding the accident cards. In all, there were 16,421 accidents, of which 60 percent were on tangents and 16 percent on curves.

Table 4 classifies the cards by severity of accident within each State. There are cards for 557 fatal accidents, 5,671 personal-injury accidents, and 10,193 property-damage accidents.

Table 4 also lists certain ratios for each State. The ratio of the total number of accidents to the number of fatal accidents is a rough measure of the completeness of accident reporting. It has its limitations, however, for while all fatal accidents are probably reported, it is most unlikely that the true total-to-fatal ratio is the same in every State. In any event, this ratio is the basis of an adjustment which is used in some of the rate computations.

The ratio of all accidents to those involving either deaths or injuries (fatal-plus-injury) has a similar interest. It might also be used for adjustments, but this has not been done so far. This ratio varies much less among the different States than does the total-to-fatal ratio. The table also shows the ratio of the number of injury accidents to the number of fatal accidents in each State, and the last column of table 4 lists the adjustment factors (obtained by dividing 50 by the total-to-fatal ratios) which are used in computing the type 1 accident rates, as explained subsequently.

Method of Analysis

It was difficult to decide how to combine the detailed data from different States. The reporting requirements vary, and it cannot be assumed that the reporting laws are fully complied with in every State. There are three essentially different ways of dealing with this problem. One way is to use a system of weights involving an adjustment factor for each State. A State which is believed to report only half its accidents would have an adjustment factor of 2, so that each reported accident would count as two adjusted accidents. This is substantially the approach that was used by McCormack and Baldwin in their earlier reports of preliminary findings from this study. The trouble with adjustments is that they give the most weight to the least reliable data, and that the adjustment factors are computed on the basis of a dubious assumption.

A second approach would dispense with adjustments but would use only the data from States whose reporting meets a certain standard. This avoids the distortions caused by the adjustment process, but it has the drawback of reducing the amount of usable data. A variation of this approach would use only the fatal accidents (in all the States), or only the fatal and injury accidents. To use only fatal accidents is impractical, however, for it would reduce the study to only 557 accidents. The use of both fatal and injury accidents has more to commend it, but there would still be a heavy reduction in the amount of data available for analysis. Moreover, it is doubtful that fatal (or fatal and injury) accidents are affected by highway features in the same way as accidents of all degrees of severity combined.

The third approach is to ignore the problem altogether and simply count the accidents in all the participating States, irrespective of the variation in reporting standards. For all its crudeness, this method turns out to be generally superior to the other two.

All three of these approaches have been used. (The choice among them is explained in the final section of the report.) In the ensuing discussion they will be called type 1 rates, type 2 rates, and type 3 rates, respectively. The type 1 rates use all the participating States, with the number of accidents in each State multiplied by the adjustment factor given in the last column of table 4. The type 2 rates do not use adjustments, and include only those States having a total-to-fatal ratio of at least 25. The type 3 rates use all the participating States, without adjustment. Use of the adjustment factors involves the assumption that the total-to-fatal ratio would be the same in every State if the reporting standards were identical. There is at least one State in the present study for which this assumption is clearly unreasonable (see discussion of Virginia, p. 177).

Once it has been decided which material to use and whether the count should be of actual or of adjusted accidents, the process of computing a rate involves (1) counting the total number of accidents on sections in a particular category, (2) adding up the total vehicle-mileage on these sections, and (3) dividing the former sum by the latter to get the rate in terms of accidents per million vehicle-miles. To study the effect of grade, for example, the highway sections are divided into several groups on the basis of their grades. Then the rates for these groups are computed and examined to see if there is a steady trend or other close relation between accident rate and grade. If the rates show indications of a trend but are somewhat irregular, it is possible to make a statistical test of whether or not a trend really exists.

The slope of the "best" straight line—the regression coefficient—is computed, with each rate weighted in proportion to the amount of travel on which it is based. Also computed are the confidence limits of this slope, from which one can tell at a glance how reliable the estimated slope is and whether or not it is significantly different from zero. These quantities are presented in those cases where they will aid in understanding the analyses.

For those to whom the correlation coefficient is more familiar than the regression

Table 5.—Accident rates on tangents, by grade and roadway type

[*rates are per million vehicle-miles*]

Grade, in percent	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided [1]		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
Less than 3	5,442	3.7	194	6.1	1,043	6.0	827	4.7	504	2.2
3–3.99	321	4.0	18	5.6	136	7.1	83	3.4	139	2.9
4–4.99	253	3.6	6	7.7	65	6.8	56	4.4	59	1.6
5–5.99	322	4.4	3	4.4	49	6.8	9	3.6	46	2.1
6–6.99	86	4.0	1	10.0	29	10.2	1	1.4	13	1.7
7 or more	49	3.9	5	14.4	26	10.1	6	15.6	13	1.5
Less than 3	5,442	3.7	194	6.1	1,043	6.0	827	4.7	504	2.2
3 or more	1,031	4.0	33	6.8	305	7.5	155	4.1	270	2.3
TYPE 2 ACCIDENT RATES (SELECTED STATES,[2] WITHOUT ADJUSTMENT)										
Less than 3	3,507	3.0	100	6.5	644	3.3	701	3.0	504	1.6
3–3.99	219	3.3	14	3.4	78	3.5	78	2.5	139	1.7
4–4.99	175	2.7	1	1.2	12	2.0	43	3.0	59	1.5
5–5.99	259	4.2	0	....	18	3.7	6	1.8	46	1.4
6–6.99	50	2.9	0	....	0	....	1	2.5	13	1.6
7 or more	48	3.7	0	....	5	6.3	0	....	13	1.5
Less than 3	3,507	3.0	100	6.5	644	3.3	701	3.0	504	1.6
3 or more	751	3.4	15	2.7	113	3.4	128	2.6	270	1.6
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
Less than 3	5,442	2.2	194	2.6	1,043	2.7	827	2.9	504	1.6
3–3.99	321	2.3	18	2.5	136	2.8	83	2.5	139	1.7
4–4.99	253	2.2	6	2.3	65	2.0	56	2.6	59	1.5
5–5.99	322	3.1	3	.9	49	2.3	9	1.8	46	1.4
6–6.99	86	2.2	1	2.0	29	2.4	1	1.4	13	1.6
7 or more	49	3.7	5	3.1	26	2.6	6	3.3	13	1.5
Less than 3	5,442	2.2	194	2.6	1,043	2.7	827	2.9	504	1.6
3 or more	1,031	2.5	33	2.2	305	2.4	155	2.5	270	1.6

[1] Excluding highways with controlled access. This applies to all the tables.
[2] States having a total-to-fatal ratio of 25 or more. This applies to all the tables.

coefficient, it should be pointed out that the two are closely related. ($b = r\sigma_y/\sigma_x$, where b is the regression coefficient, r the correlation coefficient, and σ_y and σ_x the standard deviations of y and x respectively.) The significance of the departure of b from zero is the same as that for r. The use of b has two advantages, however, over the use of r: (1) it is of more inherent interest, since it may be more important to know about a relationship of steep slope with low reliability than one of small slope with high reliability; (2) if the true relationship is a straight line, the estimated value of b is normally distributed while that of r is highly skewed. So the estimate of b is much less affected by sample size than that of r.

For study purposes the highway sections have been classified as tangents, curves, intersections, structures, and miscellaneous (railroad grade crossings, toll stations, and transitions in width of roadway or median). These are further subdivided according to the number of lanes on the study route.

Some of the material is presented in tables, some in bar graphs. Most of the tables present the three types of accident rates already described. As a guide to the reliability of the various rates, there is presented along with each rate the actual number of accidents on which it is based.

The graphs, with one exception, show only the type 3 rates, i.e., the ones that use accidents from all the States without any adjustments. On these graphs the bars differ in width according to the number of vehicle-miles on which each rate is based.

Tangents: Effect of Grade

Two- and three-lane roads

Table 5 shows how the accident rates on tangents vary with the gradient of the

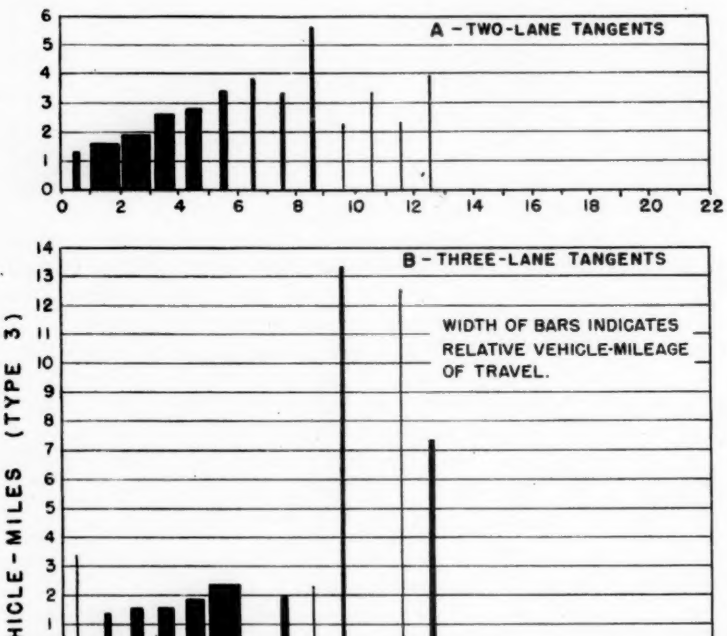

Figure 3 **(above).**—*Accident rates on tangents, by volume of traffic.*

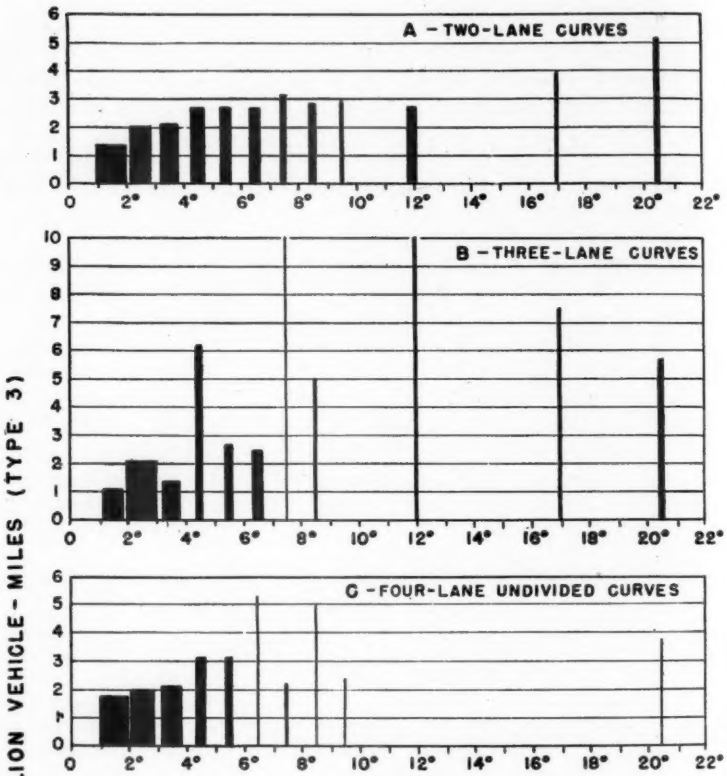

Figure 4 **(right).**—*Accident rates on curves, by degree of curvature.*

Table 6.—Accident rates on tangents, by volume of traffic and roadway type

[*rates are per million vehicle-miles*]

Average daily traffic	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
0–4,900	5,007	3.6	79	5.3	129	5.6	25	3.1	265	4.0
5,000–9,900	1,396	4.3	102	6.6	481	7.3	388	4.0	3	2.1
10,000–14,900	71	3.6	46	10.4	422	6.9	465	5.3	166	1.4
15,000 or more	0	....	0	....	317	4.1	126	5.1	340	1.5
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
0–4,900	2,868	2.9	5	2.0	18	1.4	19	1.4	265	2.0
5,000–9,900	1,320	3.8	64	4.8	117	3.2	280	2.4	3	2.1
10,000–14,900	71	3.3	46	8.1	309	3.5	380	3.5	166	1.4
15,000 or more	0	....	0	....	314	3.7	126	4.4	340	1.5
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
0–4,900	5,007	2.1	79	1.6	129	1.6	25	1.6	265	2.0
5,000–9,900	1,396	3.6	102	2.9	481	2.2	388	2.4	3	2.1
10,000–14,900	71	3.3	46	8.1	422	3.5	465	3.4	166	1.4
15,000 or more	0	....	0	....	317	3.6	126	4.4	340	1.5

Table 7.—Accident rates on two-lane tangents, by width of pavement

[*rates are per million vehicle-miles*]

Pavement width, in feet	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
16 or less	246	5.5	246	3.3	246	4.3
18	1,795	4.0	742	2.9	1,795	2.1
20	3,263	3.4	2,434	3.0	3,263	2.3
21 or 22	506	4.7	359	3.1	506	2.6
23 or 24	299	3.8	138	3.9	299	1.9
25	45	2.4	38	1.9	45	1.6
26	47	4.0	47	3.6	47	3.3
27	47	3.3	42	3.4	47	2.5
28	132	4.6	132	3.6	132	3.6
29 or more	94	3.1	81	2.8	94	2.5

highway. On two-lane roads there does not appear to be any particular relation between accident rate and grade, no matter which of the three rate types is considered. (This statement refers to the *total* correlation between accident rate and grade, ignoring all other highway features. It may be that, when the appropriate other features are held constant, there is a significant *partial* correlation. All the statements which will be made in connection with single-factor analyses refer to total correlations only.)

For each of the three types the slope (regression coefficient) is positive, i.e., the accident rate tends, on the average, to increase as the grade increases. However, none of the three slopes is significantly different from zero at the 5-percent level of significance. (The 5-percent level is widely used in statistical analyses.) This means that the amount of scatter is such that there is more than a 5-percent chance that the true slope may be zero or negative.

The three types of accident rates differ considerably. The type 1 rate is the highest, because it includes accidents which are assumed to have occurred without being reported. The type 3 rate is the lowest, because it uses only the accidents that were actually reported and includes States in which the reporting is known to be poor. The type 2 rate is in between. It might be thought that this rate would be the most reliable, because it includes only the accidents actually reported in States where reporting is presumed to be good; but it suffers from being based on a smaller sample than the other two types of rates.

A disturbing feature in all three types of rates is the large amount of apparently meaningless fluctuation. For example, the type 2 rate takes the values 3.0, 3.3, 2.7, 4.2, 2.9, and 3.7 as the grades increase steadily. These fluctuations are too large to be due to sampling variation. They cannot be a result of the adjustment process, for they are just as prominent in the unadjusted rates (types 2 and 3) as in the adjusted one; anyway, the same effect is found within individual States. Nor are they peculiar to the effect of grade; similar fluctuations occur with most other highway features. They may be due to the oversimplification caused by studying only one or two features at a time while ignoring all the rest, or they may be a result of the difficulty in obtaining absolutely accurate data. This could have the effect of obscuring relations which really exist. There is some evidence to support the latter belief. After submitting the data for the present study, Minnesota conducted another study of the same highway which in some respects parallels the work being described here. The collection of data was all new, none of the information being carried over from the earlier work. Very great care was used in checking all information, particularly with regard to the exact locations of accidents. The value of this care is demonstrated by the greater consistency of the results in their report.[2]

To sum up, the accident rate on two-lane tangents does not appear to be significantly affected by grade.

On three-lane tangents, as on the two-lane tangents, most of the travel was on roads of less than 3-percent grade. Large fluctuations are present in the accident rates, and no reliable relation is found between accident rate and the percentage of grade. The slopes for the types 1 and 3 rates lack statistical significance, as with the two-lane roads. The type 2 rate has a significant negative slope, with the rate declining as the grades become steeper; but the decline is meaningless because of a peculiar distribution of travel among the different States. The high rate for roads of less than 3-percent grade is caused entirely by the figures from one State, Oregon, which included no roads with higher grades. If Oregon data are excluded from the type 2 rate, the significant relation disappears.

Four-lane roads

On four-lane tangents without a median, it seems unlikely that the grade has any effect on the accident rate, even though the type 1 rate does have a significant upward slope of 0.69 ± 0.48 per percent of grade. (This means that the rate increases by about 0.69 for each 1-percent increase in the grade. There is a 95-percent chance that the true slope is between $0.69 - 0.48 = 0.21$ and $0.69 + 0.48 = 1.17$). The type 3 rate, which uses the very same accidents and vehicle-mileage, tends to become smaller as the grades increase but in a way that does not indicate a statistically significant trend. The type 2 rate is irregular but shows a slight tendency to increase as the grades increase.

This is an example of how each type of accident rate can point to a different conclusion. It appears that grades in the range used on main rural highways do not—when other factors are ignored—have any appreciable effect on the accident rate for four-lane undivided tangents, especially in view of the fact that the significant rate of increase for the type 1 rate is the result of a peculiar circumstance which distorts the true picture. Virginia, whose adjusted accident rate is much higher than that of any of the other States, contributes increasing proportions of the total travel as the grades

[2] *Minnesota Rural Trunk Highway Accident, Access Point and Advertising Sign Study* (1951).

Table 8.—Accident rates on two-lane tangents, by width of shoulders

[*rates are per million vehicle-miles*]

Shoulder width, in feet	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
Curb	10	2.9	3	0.8	10	1.4
0–4.9	2,673	3.9	2,012	3.1	2,673	2.6
5–7.9	2,789	3.6	1,556	3.1	2,789	2.0
8–9.9	525	3.6	338	3.0	525	2.4
10 or more	476	4.1	350	3.3	476	2.8

Table 9.—Accident rates (type 3) on two-lane tangents, by volume of traffic, pavement width, and shoulder width

[*rates are per million vehicle-miles*]

Pavement width, in feet	Number of accidents and accident rate when shoulder width is—									
	Less than 5 feet		5 to 7.9 feet		8 to 9.9 feet		10 feet or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY										
16 or less	97	2.8	96	5.2	0	....	1	3.3	194	3.6
18	679	2.0	871	2.1	82	2.0	58	3.8	1,690	2.1
20	991	2.7	1,027	1.6	223	1.8	66	1.6	2,307	2.0
21–22	193	2.4	25	1.4	65	2.5	117	2.9	400	2.4
23–24	67	2.5	167	1.5	16	2.1	9	2.7	259	1.7
25 or more	35	2.3	61	1.6	0	....	55	2.6	151	2.1
Total	2,062	2.4	2,247	1.8	386	2.0	306	2.5	5,001	2.1
5,000–9,900 VEHICLES PER DAY										
16 or less	4	4.0	48	20.0	0	....	0	....	52	15.3
18	14	1.7	73	3.5	12	2.0	2	2.5	101	2.8
20	469	3.5	237	3.0	104	7.9	71	2.9	881	3.5
21–22	49	3.0	29	2.7	23	11.5	5	1.7	106	3.3
23–24	13	9.3	27	7.3	0	....	0	....	40	6.8
25 or more	33	3.9	88	2.9	0	....	92	4.4	213	3.6
Total	582	3.4	502	3.4	139	6.3	170	3.5	1,393	3.6
ALL VOLUMES										
16 or less	101	2.8	144	7.0	0	....	1	3.3	246	4.3
18	693	2.0	944	2.1	94	2.0	60	3.7	1,791	2.1
20	1,497	2.9	1,297	1.8	327	2.4	137	2.1	3,258	2.3
21–22	242	2.5	54	1.9	88	3.2	122	2.8	506	2.6
23–24	80	2.8	194	1.7	16	1.9	9	2.7	299	1.9
25 or more	68	2.8	150	2.2	0	....	147	3.5	365	2.7
Total	2,681	2.6	2,783	2.0	525	2.4	476	2.8	6,465	2.2

Table 10.—Accident rates on two-lane tangents, by frequency of curves

[*rates are per million vehicle-miles*]

Number of curves per mile	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
0–0.4	1,251	4.0	442	3.1	1,251	1.7
0.5–0.9	1,463	3.9	649	3.4	1,463	2.1
1.0–1.4	580	3.4	329	2.7	580	2.0
1.5–1.9	771	3.4	573	2.4	771	2.2
2.0–2.9	588	4.2	492	3.8	588	3.1
3.0–3.9	552	3.1	508	3.0	552	2.6
4.0–4.9	806	3.8	806	3.5	806	3.5
5.0–5.9	405	3.2	405	3.0	405	3.0
6.0–6.9	55	5.5	55	4.3	55	4.3
7 or more	0	....	0	....	0	....

increase. This makes the type 1 rate appear to increase, even though there is no increase when Virginia is considered by itself or when all the other States are considered with only Virginia excluded (see further discussion of Virginia, p. 177).

For four-lane tangents having a median but no control of access, the accident rates are also inconclusive. None of the slopes is statistically significant, but there is some tendency for the rate to decline as the grade increases. On four-lane divided roads with controlled access, too, the grade has no particular effect on the accident rates.

In summary, on tangent highway sections there does not appear to be any relation between grade and accident rates. In these analyses the roads have been classified only by grade, so it remains possible that grade may have some effect on the accident rate when the appropriate other features are held constant.

Tangents: Effect of Volume

Two- and three-lane roads

Figure 3A shows how the accident rate on two-lane tangents varies with the average daily traffic when all other characteristics of the highway section are ignored. This is a typical example of the type of bar graph used in this report. Each bar conveys three distinct pieces of information. The height of the bar indicates the type 3 accident rate for the set of roads which the bar represents. The horizontal position of the bar indicates the average daily traffic volume on the roads represented. The width (thickness) of the bar indicates the number of vehicle-miles of travel on which the rate is based. No scale is shown for these widths, since it is only the relative widths that matter. The scales are different in each graph.

To be specific, the first bar in figure 3A shows that the type 3 rate is 1.3 for roads carrying from 0 to 900 vehicles per day. The next bar shows the rate to be 1.6 for roads carrying from 1,000 to 1,900 vehicles per day, and that this rate is based on about five times as much experience as the first rate. And so on.

The graph suggests a definite pattern, which in fact is the same for all three types of rates. (Information concerning all three types of rates is presented in table 6.) The accident rate increases steadily with increasing volume, reaching a maximum for roads carrying 8,000 to 9,000 vehicles per day. Heavier traffic reduces the accident rate somewhat, presumably because the extreme congestion at such high volumes makes it difficult for drivers to engage in passing maneuvers. The latter point is of small interest to the highway designer, who would hardly recommend two-lane construction for a road expected to carry as many as 9,000 vehicles per day.

In the range that is of principal interest the relation is simple and straightforward: higher traffic volumes mean higher accident rates.

There is a similar increase on three-lane tangents, as shown in figure 3B. The type 1 and type 3 rates both increase significantly as the traffic volume becomes larger. The information for the type 2 rate is too fragmentary to be of much value.

Four-lane roads

Figure 3C represents the condition on four-lane undivided tangents. All three types of accident rates have a pattern similar to that for the two-lane tangents: the rate goes up until a certain volume is reached, after which it drops down again. But the three types of rates do not have

their maxima at the same traffic volume. The type 1 rate reaches its peak between 5,000 and 10,000 vehicles per day, while the type 2 and type 3 rates are highest for volumes between 15,000 and 20,000. The type 1 rate would have its peak in this same range if the Virginia figures were omitted. The Virginia data play a disturbing role in many of the type 1 rates. They have a low total-to-fatal ratio, with the consequently high adjustment factor of 4.59. Yet this adjustment factor seems excessive, for the adjusted accident rates are usually much higher for Virginia than for the other States. For example, on four-lane undivided tangents carrying between 5,000 and 9,900 vehicles per day, the adjusted accident rate for Virginia is 10.4 while it is only 3.7 for all the other States combined. Both rates are based on more than 200 accidents.

Figure 3D shows the same information for four-lane divided tangents without controlled access. The pattern is the same as before, with the accident rate going up as traffic volume increases. If there is any volume above which the accident rate begins to drop, it is beyond the range of these data, for which the maximum volume is 20,000 vehicles per day. The type 1 rate is higher in the 10,000 to 14,900 group than in the 15,000 to 19,900 group, but this is due to the peculiar effect of Virginia discussed previously.

The accident rates for four-lane divided roads with controlled access are shown in figure 3E. The rates appear to be somewhat lower for high volumes than for low volumes, but the appearance is misleading. The volumes under 5,000 come almost exclusively from Pennsylvania, while the volumes over 5,000 are all from Connecticut. The comparison is not so much between low volumes and high volumes as between Pennsylvania and Connecticut. Pennsylvania's accident rate is higher than Connecticut's, even though the average volumes are 4,000 and 15,000 respectively. Examination of the trend within each State shows that in Pennsylvania, where the volumes range from 3,000 to 5,000 vehicles per day, the accident rate increases steadily with increasing volume, but it is hard to draw conclusions from such a small range of volumes. In Connecticut the range is wide, and there is no significant trend. The evidence at hand does not indicate that traffic volume has any particular effect on the accident rate on four-lane divided tangents with controlled access.

Summary

The foregoing material is summarized in table 6. In most cases the average daily traffic has a considerable effect on the accident rate on tangent highway sections. The common pattern is for the accident rate to increase as the volume increases. At very high volumes the accident rate usually drops somewhat, probably because of congestion.

Table 11.—Accident rates (type 3) on two-lane tangents, by volume of traffic, frequency of curves, and length of tangent

[rates are per million vehicle-miles]

Number of curves per mile	Number of accidents and accident rate when length of tangent is—									
	Less than 1 mile		1 to 1.9 miles		2 to 2.9 miles		3 miles or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day										
Less than 0.5	66	1.9	64	1.5	135	2.5	881	1.6	1,146	1.7
0.5–0.9	213	1.9	279	2.2	197	2.0	466	1.5	1,155	1.8
1–1.9	492	2.0	232	1.7	75	1.5	231	2.6	1,030	2.0
2–2.9	342	2.7	51	2.8	4	1.0	7	3.3	404	2.7
3 or more	1,071	2.9	95	2.4	26	2.8	26	4.9	1,218	2.9
Total	2,184	2.5	721	2.0	437	2.0	1,611	1.7	4,953	2.0
5,000–9,900 Vehicles per Day										
Less than 0.5	16	8.4	0	....	0	....	68	4.0	84	4.4
0.5–0.9	88	4.7	17	3.5	119	4.9	63	3.8	287	4.5
1–1.9	190	2.9	33	1.4	0	....	45	3.8	268	2.6
2–2.9	128	4.1	51	5.3	0	....	0	....	179	4.3
3 or more	408	3.2	93	4.8	19	5.3	31	8.4	551	3.6
Total	830	3.4	194	3.4	138	4.8	207	4.2	1,369	3.6
All Volumes										
Less than 0.5	82	2.2	64	1.5	135	2.5	949	1.6	1,230	1.7
0.5–0.9	301	2.3	296	2.2	316	2.6	529	1.6	1,442	2.0
1–1.9	714	2.2	265	1.7	75	1.5	276	2.8	1,330	2.1
2–2.9	470	3.0	102	3.7	4	1.0	7	3.3	583	3.1
3 or more	1,488	2.9	188	3.7	73	4.1	57	6.3	1,806	3.1
Total	3,055	2.7	915	2.2	603	2.4	1,818	1.8	6,391	2.3

Table 12.—Accident rates (type 3) on two-lane tangents, by volume of traffic, frequency of intersections, and frequency of structures

[rates are per million vehicle-miles]

Number of intersections per mile	Number of accidents and accident rate when the number of structures per mile is—					
	Less than one		One or more		Total	
	Number	Rate	Number	Rate	Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day						
Less than 0.5	357	1.8	19	1.9	376	1.8
0.5–0.9	798	1.9	47	1.3	845	1.8
1–1.9	2,425	1.9	280	2.2	2,705	1.9
2–2.9	675	2.8	28	7.6	703	2.9
3 or more	359	2.7	13	5.9	372	2.7
Total	4,614	2.0	387	2.2	5,001	2.1
5,000–9,900 Vehicles per Day						
Less than 0.5	23	5.0	0	....	23	5.0
0.5–0.9	23	11.5	0	....	23	11.5
1–1.9	382	4.3	87	4.0	469	4.2
2–2.9	553	3.5	48	20.0	601	3.8
3 or more	278	2.5	0	....	278	2.5
Total	1,259	3.5	135	5.6	1,394	3.6
All Volumes						
Less than 0.5	380	1.9	19	1.9	399	1.9
0.5–0.9	821	1.9	47	1.3	868	1.9
1–1.9	2,835	2.1	367	2.5	3,202	2.1
2–2.9	1,262	3.1	76	12.5	1,338	3.2
3 or more	646	2.6	13	5.9	659	2.6
Total	5,944	2.2	522	2.6	6,466	2.3

As between the different types of roads at the same volumes, the conclusions depend on which type of accident rate is examined. Judged by the type 1 rate, the safest roads at volumes below 10,000 vehicles per day are the four-lane divided roads without controlled access, followed closely by the two-lane roads; the four-lane undivided roads are the worst in this volume range. Above 10,000 vehicles per day the four-lane divided roads with controlled access are far and away the safest, while the three-lane roads have much the highest accident rates.

The type 3 rates show little difference between road types for volumes under 5,000

Table 13.—Accident rates on two-lane tangents, by frequency of roadside establishments (including dwellings)

[*rates are per million vehicle-miles*]

Number of establishments per mile	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
0–0.9						
1.0–4.9	650	4.3	220	5.3	650	1.8
5.0–9.9	2,131	3.4	904	2.6	2,131	1.8
10.0–19.9	1,567	4.4	1,205	3.3	1,567	2.9
20.0–49.9	1,770	3.5	1,637	3.2	1,770	2.9
50 or more	356	4.0	293	2.9	356	3.1
	0	....	0	....	0	....

Table 14.—Accident rates on two-lane tangents, by frequency of sight-distance restrictions

[*rates are per million vehicle-miles*]

Number of restrictions per mile	Number of accidents and accident rate for—					
	Type 1 accident rate (all States, adjusted)		Type 2 accident rate (selected States, unadjusted)		Type 3 accident rate (all States, unadjusted)	
	Number	Rate	Number	Rate	Number	Rate
0–0.9	3,472	3.7	1,833	2.8	3,472	2.0
1.0–1.9	1,061	4.3	588	4.0	1,061	2.5
2.0–2.9	891	4.1	811	3.4	891	3.1
3.0–3.9	684	3.3	661	3.0	684	3.0
4.0–4.9	354	3.1	354	2.9	354	3.0
5.0–5.9	12	2.7	12	2.7	12	2.7

vehicles per day, while the two-lane roads have the *highest* type 3 rate for volumes between 5,000 and 10,000. Above 10,000 the conclusion is the same as before: the three-lane roads are the most hazardous, while the controlled-access four-lane divided roads are the safest.

The type 2 rates are still different. Below 5,000 vehicles per day information is fragmentary. Between 5,000 and 10,000, the three-lane roads are the worst, while the four-lane divided roads without controlled access are the safest of those for which the samples are adequate. Above 10,000 vehicles per day the conclusion is the same as for the types 1 and 3 rates.

Two-Lane Tangents: Effects of Other Features

Since traffic volume has a pronounced effect on the accident rates, it is desirable to group the roadway sections by volume before studying the effects of other factors. Alternatively, the volume itself can be made one of the independent variables in a multiple regression analysis, a procedure which separates the effects of different factors. Both approaches have been used.

With grade and volume as independent variables, the multiple analysis corroborates the earlier conclusion that grade has no statistically significant effect on the accident rate, while volume does.

Pavement width, shoulder width, and traffic volume

If the two-lane tangents are classified solely according to their pavement width—irrespective of traffic volume, shoulder width, or other factors—the results are as shown in table 7. The evidence is confusing. It is not even clear whether 24-foot pavements are safer than 20-foot pavements; the type 3 rate suggests that they are, while the types 1 and 2 rates indicate that they are not. Neither is it definitely established that very narrow pavements have the highest accident rates. The types 1 and 3 rates indicate that they do, but the type 2 rate shows pavements of 16 feet as having a lower accident rate than those of 24 feet.

Similar information about the effect of shoulder width is presented in table 8. There is no indication that shoulder width, considered alone, has any bearing on the accident rates.

With the roads grouped according to traffic volume in 5,000 vehicle-per-day intervals, a multiple analysis has been made in each group, using pavement width and shoulder width as the independent variables. The complete table is too complicated for inclusion here, but a condensed version is given in table 9.

None of the effects is statistically significant. In neither the 0 to 4,900 volume group nor the 5,000 to 9,900 group is there a statistically significant effect on either the type 1 or the type 3 accident rate due to (1) pavement width, for constant shoulder width; (2) shoulder width, for constant pavement width; or (3) pavement width and shoulder width acting together. (For (1) and (2) the statistical tests are t-tests of the partial regression coefficients. For (3) they are F-tests of the multiple correlation coefficients. The two types of tests are equivalent when there is only one independent variable.) The material in this study indicates that neither pavement width nor shoulder width nor any combination of them has a determinable effect on the accident rates on two-lane tangents.

Frequency of curves

In the belief that driver behavior and accident experience might be affected by the frequencies of occurrence of curves, intersections, and other such features, the study routes have been divided into "frequency sections" averaging 10 to 15 miles in length. Every card contains all the frequency information for the frequency section to which it belongs.

Table 10 shows how the accident rates on two-lane tangents vary with the average number of curves per mile. As usual, the result depends on which figures are examined. The only rates with a significant trend are the type 3 rates, which go up as the curves become more frequent. The types 1 and 2 rates suggest an opposite con-

Table 15.—Accident rates (type 3) on two-lane tangents, by volume of traffic, percentage of commercial traffic, and percentage of night traffic

[*rates are per million vehicle-miles*]

Commercial traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is night traffic is—							
	0 to 19 percent		20 to 29 percent		30 to 39 percent		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TRAFFIC VOLUME 0 TO 4,900 VEHICLES PER DAY								
0–9.9	1	5.0	0	....	108	2.8	109	2.8
10–14.9	786	3.0	281	2.1	503	2.7	1,570	2.7
15–19.9	249	3.7	564	1.6	470	2.5	1,283	2.1
20–24.9	8	1.3	866	2.2	850	1.4	1,724	1.7
25 or more	0	....	221	1.7	94	1.3	315	1.6
Total	1,044	3.1	1,932	1.9	2,025	1.8	5,001	2.1
5,000–9,900 VEHICLES PER DAY								
0–9.9	0	....	0	....	68	2.5	68	2.5
10–14.9	303	6.4	183	4.2	194	2.8	680	4.2
15–19.9	204	6.8	0	....	235	2.7	439	3.8
20–24.9	0	....	72	3.5	111	2.3	183	2.7
25 or more	0	....	9	1.4	15	1.9	24	1.7
Total	507	6.5	264	3.8	623	2.6	1,394	3.6
ALL VOLUMES								
0–9.9	1	5.0	0	....	176	2.7	177	2.7
10–14.9	1,117	3.6	464	2.6	697	2.8	2,278	3.1
15–19.9	453	4.7	564	1.6	739	2.6	1,756	2.4
20–24.9	8	1.3	938	2.3	970	1.5	1,916	1.8
25 or more	0	....	230	1.7	109	1.4	339	1.6
Total	1,579	3.8	2,196	2.0	2,691	2.0	6,466	2.3

clusion, that the tangent sections interspersed with one or two curves per mile are safer than those in places where curves are quite rare. Either conclusion, once established, has a plausible explanation, but the figures are confusing. Even the simple fact that all three types of rates have their highest values for curve frequencies of six or more curves per mile is not so simple as it seems, for the State which supplied all these sections, Oregon, has a still higher accident rate for frequencies between 4.0 and 4.9.

Table 11 gives a multiple breakdown of the type 3 accident rates by curve frequency, tangent length, and traffic volume. The length is that of the entire tangent, even though it may be broken up into a number of shorter sections by the presence of minor intersections or structures. The multiple regression analysis corroborates the confusing conclusions from the analysis of curve frequency alone. In the lowest volume group, the effect of adding curves is to reduce the type 1 accident rate and to increase the type 3 rate. There is no significant effect at higher volumes.

The value of the multiple analysis becomes apparent when we examine the effect of tangent length on the type 3 accident rates for volumes under 5,000 vehicles per day. The totals for all curve frequencies combined indicate a high positive correlation between accident rate and tangent length. Even when the detailed breakdown is used, the simple correlation with tangent length is still statistically significant. Yet it falsifies the truth. For there is a large negative correlation between tangent length and curve frequency—i.e., long tangents have a strong tendency to be associated with low curve frequencies. To determine the effect of different tangent lengths on roads having the same curve frequency we must use the *partial* correlation coefficient of accident rate with tangent length. This coefficient is not statistically significant for any volume group.

Frequency of other features

A breakdown of accident rates according to structure frequency, intersection frequency, and traffic volume is given in table 12. There are no statistically significant effects of structure frequency or intersection frequency. However, there is some tendency for the type 3 rates to rise with increasing intersection frequency when the traffic volume is low, and to fall with increasing intersection frequency when the traffic volume is high.

Table 13 shows the effect on the accident rates of the frequency of roadside establishments (including dwellings). The results are perplexing. Only the type 3 rate comes anywhere near showing a significant trend; these figures suggest that adding roadside establishments makes a road more hazardous. The type 2 rate, on the other hand, seems to indicate that the only roads that are particularly bad are those having less

Table 16.—Accident rates on curves, by degree of curvature and roadway type

[rates are per million vehicle-miles]

Curvature, in degrees	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
0–2.9	504	2.6	11	5.6	98	4.9	95	2.4	180	2.4
3–5.9	596	3.6	11	9.8	90	8.4	65	4.2	162	3.4
6–9.9	338	3.6	6	14.1	16	7.9	5	11.9	38	5.6
10 or more	354	4.8	11	28.0	3	5.8	12	30.6	0	....
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
0–2.9	340	1.8	0	....	43	1.9	33	0.7	180	1.6
3–5.9	447	2.5	0	....	33	2.1	52	2.7	162	2.3
6–9.9	287	2.9	0	....	10	2.9	1	1.2	38	4.5
10 or more	281	3.4	1	10.0	0	....	0	....	0	....
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
0–2.9	504	1.6	11	1.7	98	1.9	95	1.8	180	1.6
3–5.9	596	2.5	11	2.8	90	2.6	65	2.4	162	2.3
6–9.9	338	2.8	6	3.5	16	3.3	5	3.1	38	4.5
10 or more	354	3.5	11	7.3	3	1.2	12	6.7	0	....

Table 17.—Accident rates on tangents and curves,[1] by roadway type

[rates are per million vehicle-miles]

Location	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
Tangents	6,474	3.7	227	6.1	1,348	6.4	982	4.6	774	2.2
Curves	1,794	3.3	39	10.2	210	6.5	177	3.8	380	2.9
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
Tangents	4,259	3.1	115	5.3	757	3.3	829	2.9	774	1.7
Curves	1,355	2.5	1	2.5	86	1.9	86	1.3	380	2.0
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
Tangents	6,474	2.3	227	2.5	1,348	2.7	982	2.9	774	1.7
Curves	1,794	2.3	39	2.8	210	2.2	177	2.1	380	2.0

[1] All volumes, grades, curvatures, etc.

Table 18.—Accident rates on curves, by volume of traffic and roadway type

[rates are per million vehicle-miles]

Average daily traffic	Accidents on two-lane roads		Accidents on three-lane roads		Accidents on four-lane roads					
					Undivided		Divided		Controlled access	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)										
0–4,900	1,387	3.5	21	9.1	25	5.7	1	1.1	140	3.8
5,000–9,900	403	3.0	18	11.7	96	7.6	43	4.4	0	....
10,000–14,900	4	0.6	0	....	69	3.4	117	4.1	45	1.8
15,000 or more	0	....	0	....	20	1.9	27	6.5	63	1.4
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)										
0–4,900	957	2.5	1	2.5	2	0.6	0	....	140	1.9
5,000–9,000	394	2.7	0	....	20	1.5	18	0.7	0	....
10,000–14,900	4	0.6	0	....	34	2.0	111	2.9	45	1.8
15,000 or more	0	....	0	....	20	1.8	27	5.9	63	1.3
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)										
0–4,900	1,387	2.3	21	2.6	25	1.7	1	0.3	140	1.9
5,000–9,900	403	2.7	18	3.1	96	2.3	43	1.4	0	....
10,000–14,900	4	0.6	0	....	69	2.4	117	2.9	45	1.8
15,000 or more	0	....	0	....	20	1.8	27	5.9	63	1.3

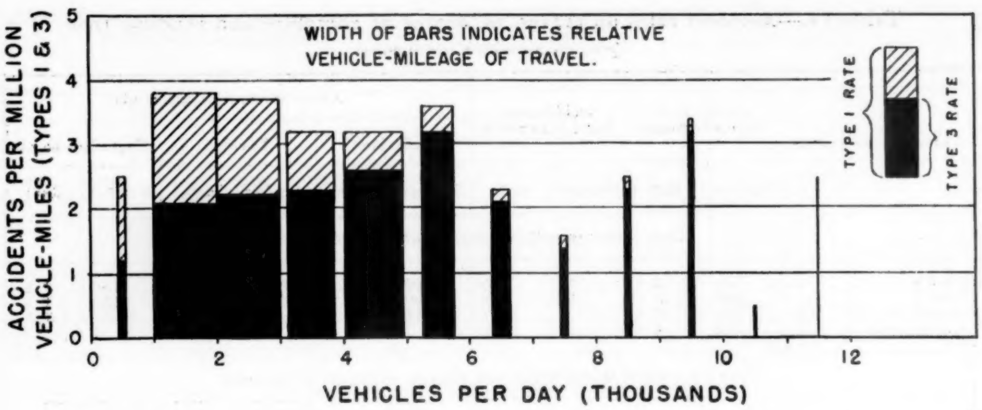

Figure 5.—Accident rates on two-lane curves (all degrees), by volume of traffic.

than one establishment per mile. The type 1 rate is quite irregular. The effect on the accident rates of the frequency of roadside establishments has been studied in more detail by the States of Minnesota and Michigan.[3] The Minnesota study is cited in footnote 2 (p. 175).

In studying the frequency of sight-distance restrictions a restriction has been defined as a stretch of road where the sight distance is less than 600 feet in flat or rolling terrain, or less than 400 feet in mountainous terrain. The relation of accident rates to the frequency of sight restrictions is shown in table 14. The type 3 rates are the most meaningful. Their slope is statistically significant, with the accident rate rising as the restriction frequency increases from zero up to about three restrictions per mile. The types 1 and 2 rates have maxima when there are between one and two restrictions per mile; they drop steadily as the frequency of restrictions increases above this number.

[3] Michigan study, *Accident Experience in Relation to Road and Roadside Features* (1952).

Commercial and night traffic

Table 15 presents a three-way breakdown of the type 3 accident rates by traffic volume, the percentage that is commercial traffic, and the percentage of the traffic that flows after dark. The multiple analysis indicates that the accident rate is reduced as the percentage of night traffic increases, when the percentage of commercial traffic remains the same; and that the accident rate also falls off with increasing commercial traffic when the night traffic is held fixed. Both these effects were unexpected.

In summary, of all the characteristics studied for their effects on the accident rate on two-lane tangents, traffic volume is

Table 19.—State-by-State accident rates (type 1) on two-lane curves, by volume of traffic

[*rates are per million vehicle-miles*]

State	Number of accidents, amount of travel, and accident rate when traffic volume is— 0 to 4,900 vehicles per day: Number of accidents (adjusted)	Travel, in million vehicle-miles	Accident rate	5,000 to 9,900 vehicles per day: Number of accidents (adjusted)	Travel, in million vehicle-miles	Accident rate
Colorado	238	71.2	3.3	12	1.3	9.2
Connecticut (1941)	269	95.1	2.8	220	80.9	2.7
Connecticut (1946)	282	112.0	2.5	69	29.2	2.4
Georgia	106	13.7	7.7	0	0	....
Iowa	220	40.5	5.4	0	1.4	....
Louisiana	56	27.4	2.0	2	0.2	10.0
Minnesota	41	33.8	1.2	0	0	....
Nebraska	201	54.7	3.7	0	0.4	....
New Mexico	50	5.0	10.0	0	0	....
Oregon	108	46.7	2.3	17	8.7	2.0
Utah	206	31.6	6.5	7	1.9	3.7
Virginia	23	3.0	7.7	0	0	....
Washington	294	63.2	4.7	120	27.0	4.4
Wisconsin	11	10.7	1.0	0	0	....
Wyoming	14	4.4	3.2	0	0	....
Total	2,119	613.0	3.5	447	151.0	3.0

Table 20.—Accident rates on two-lane curves, by volume of traffic and degree of curvature

[*rates are per million vehicle-miles*]

Curvature, in degrees	Number of accidents and accident rate when traffic volume is— 0 to 4,900 vehicles per day: Number	Rate	5,000 vehicles per day or more: Number	Rate
Type 1 Accident Rates (All States, Adjusted)				
0–2.9	395	2.7	111	2.1
3.0–5.9	423	3.7	173	3.4
6.0 or more	569	4.4	123	3.1
Type 2 Accident Rates (Selected States, without Adjustment)				
0–2.9	231	1.8	109	2.0
3.0–5.9	278	2.3	169	3.1
6.0 or more	448	3.2	120	2.9
Type 3 Accident Rates (All States, without Adjustment)				
0–2.9	395	1.6	111	1.9
3.0–5.9	423	2.3	173	3.1
6.0 or more	569	3.2	123	2.8

Table 21.—Accident rates (type 3) on two-lane curves, by volume of traffic, degree of curvature, and grade

[*rates are per million vehicle-miles*]

Curvature, in degrees	Number of accidents and accident rate when grade is— Less than 3 percent: Number	Rate	3 percent or more: Number	Rate	Total: Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day						
0–2.9	317	1.4	78	2.0	395	1.6
3–5.9	317	2.3	106	2.4	423	2.3
6–9.9	194	3.0	69	2.3	263	2.8
10 or more	155	3.4	150	3.8	305	3.6
Total	983	2.1	403	2.7	1,386	2.3
5,000–9,900 Vehicles per Day						
0–2.9	86	1.9	22	2.9	108	2.0
3–5.9	117	2.8	55	4.1	172	3.2
6–9.9	51	2.6	22	3.1	73	2.7
10 or more	27	2.5	22	3.9	49	3.0
Total	281	2.4	121	3.6	402	2.7
All Volumes						
0–2.9	405	1.6	100	2.2	505	1.6
3–5.9	434	2.4	161	2.8	595	2.5
6–9.9	245	2.9	93	2.5	338	2.8
10 or more	182	3.2	172	3.8	354	3.5
Total	1,266	2.2	526	2.8	1,792	2.3

the only one whose effect is entirely clear. The effects of the frequencies of curves, intersections, roadside establishments, and sight restrictions are all uncertain, while grade, pavement width, shoulder width, tangent length, and frequency of structures do not have any independent effects on the accident rates. The effects of commercial traffic and night traffic are inconclusive.

Curves: Effects of Curvature and Volume

Degree of curvature

The type 3 accident rates on two-lane curves, by degree of curvature, are presented in figure 4A (p. 174), and all three types of accident rates are given in table 16. The relation is clearcut: the sharper the curve, the higher the accident rate. As a matter of fact, the slopes are highly significant for all three types of rate. In accident-rate units per degree, the slopes are 0.19 ± 0.07 for the type 1 rate, 0.12 ± 0.05 for the type 2, and 0.16 ± 0.05 for the type 3. Thus, whichever type of accident rate is used, the number of accidents per million vehicle-miles increases by about 0.15 for each additional degree of curvature.

For three-lane roads, too, there seems to be a steady increase in hazard with increasing curvature, though the data are somewhat sparse. Figure 4B shows the rates based on a total of 39 accidents.

Figure 4C gives the corresponding information for four-lane undivided curves. The type 3 rate has the same upward trend as on the two- and three-lane roads, although the slope is not statistically significant; the types 1 and 2 rates are more irregular.

On four-lane divided roads the accident rate also increases as the curves become sharper. The rates are presented in figure 4D.

Figure 4E shows the relation of curvature to accident rate for four-lane divided roads with controlled access. As before, the trend is statistically significant, with the accident rates increasing by about 0.4 for each additional degree of curvature.

In summary, there is a direct relation between curvature and accident rate on all types of highways. Sharp curves have high accident rates, gradual curves have low accident rates, in-between curves have in-between accident rates.

Among different types of roads with the same degree of curvature, the data do not indicate any consistent relation.

Table 17 compares tangents with curves on each type of roadway. (These rates are for all the tangent and curve sections included in the study, irrespective of other factors.) There is no clear superiority one way or the other. The type 3 rates, which are the most consistent, indicate that tangents and curves are equally safe on two-lane roads. Tangents are a little safer than curves on three-lane and controlled-access four-lane divided roads, while they are somewhat more hazardous on the four-lane roads lacking control of access. None of these differences is large enough to warrant any strong conclusions.

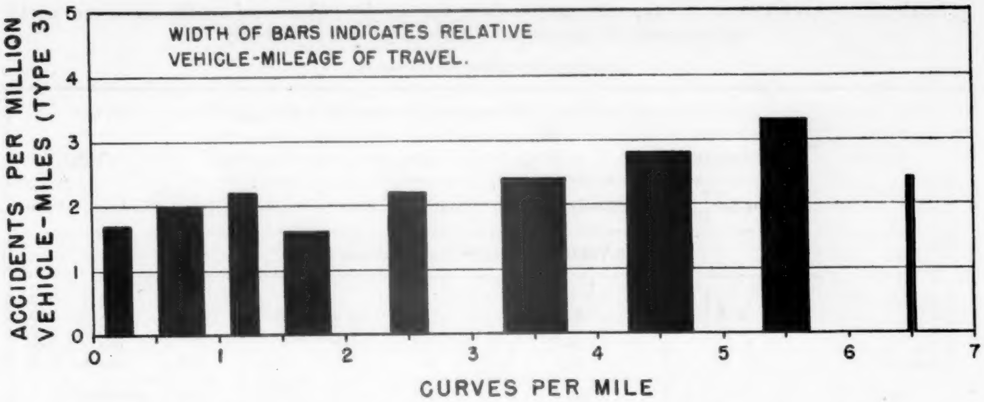

Figure 6.—Accident rates on two-lane curves (all degrees), by frequency of curves.

Effect of traffic volume

Table 18 and figure 5 show how the accident rate on two-lane curves varies with the average daily volume of traffic. The type 1 rate has a statistically significant tendency to become *smaller* as the traffic increases. The types 2 and 3 rates do not vary significantly but have a slight tendency to increase with increasing traffic.

The decline shown by the type 1 rate was unexpected, but the conclusion which it suggests is almost certainly correct. Table 19 gives a State-by-State breakdown of these rates. This table shows that every State having more than six accidents (actual, not adjusted) at volumes over 5,000 vehicles per day has a lower accident rate for these volumes than for volumes below 5,000. It has also been proved by multiple regression analysis that this decline is not a hidden effect of the degree of curvature.

Why should the accident rate decline with increasing traffic on two-lane curves, when it goes the opposite way on two-lane tangents? Analysis of the types of collisions at different volumes indicates that the distribution of accident types is practically identical for all volume groups. So one can only speculate. Perhaps the extra alertness required for driving on narrow curved roads in heavy traffic is what pulls the accident rate down. A similar decline in accident rate with increasing traffic volume is found at two-lane intersections, where the same alertness factor may be involved. It is not found at curves or intersections on wider roads.

For three-lane curves the data are pretty meager (see table 18), but such evidence as there is points to a positive correlation between accident rate and traffic volume.

On four-lane undivided curves each type of rate increases to a maximum and then falls off gradually as the traffic volume increases. The type 1 rate has its maximum between 5,000 and 10,000 vehicles per day, the type 2 has its maximum between 15,000 and 20,000, the type 3 has its maximum between 10,000 and 15,000. This is similar to the pattern for four-lane undivided tangents.

Table 22.—Accident rates on two-lane curves, by degree of curvature and frequency of curves

[rates are per million vehicle-miles]

Number of curves per mile	Number of accidents and accident rate when curvature is—							
	0 to 2.9 degrees		3 to 5.9 degrees		6 to 9.9 degrees		10 degrees or more	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate
TYPE 1 ACCIDENT RATES (ALL STATES, ADJUSTED)								
0–0.9	128	3.0	110	5.4	13	4.2	31	8.9
1.0–2.9	178	2.3	163	3.7	96	4.5	53	4.2
3.0–4.9	125	2.1	223	2.9	170	3.3	139	4.3
5.0–6.9	75	3.3	100	3.2	59	2.8	130	4.6
TYPE 2 ACCIDENT RATES (SELECTED STATES, WITHOUT ADJUSTMENT)								
0–0.9	42	1.6	47	3.2	2	1.1	4	1.4
1.0–2.9	105	1.4	97	2.1	65	2.9	30	2.6
3.0–4.9	118	2.0	203	2.5	161	3.2	117	3.3
5.0–6.9	75	3.1	100	2.9	59	2.6	130	3.9
TYPE 3 ACCIDENT RATES (ALL STATES, WITHOUT ADJUSTMENT)								
0–0.9	128	1.4	110	2.7	13	2.0	31	4.3
1.0–2.9	178	1.4	163	2.1	96	2.9	53	2.6
3.0–4.9	125	1.9	223	2.5	170	2.9	139	3.4
5.0–6.9	75	3.1	100	2.9	59	2.6	130	3.9

Table 23.—Accident rates (type 3) on two-lane curves, by volume of traffic, frequency of curves, and frequency of sight-distance restrictions

[rates are per million vehicle-miles]

Number of curves per mile	Number of accidents and accident rate when number of restrictions per mile is—									
	Less than 1		1 to 1.9		2 to 2.9		3 or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day										
Less than 0.5	76	1.7	7	1.9	0	....	3	7.5	86	1.7
0.5–0.9	138	1.8	33	3.2	3	0.9	0	....	174	1.9
1–1.9	182	1.8	40	1.7	3	0.9	0	....	225	1.7
2–2.9	38	1.9	98	2.4	16	1.7	0	....	152	2.1
3 or more	9	0.6	41	3.8	181	2.7	518	2.9	749	2.8
Total	443	1.7	219	2.4	203	2.5	521	2.9	1,386	2.3
5,000–9,900 Vehicles per Day										
Less than 0.5	1	1.7	0	....	0	....	0	....	1	1.7
0.5–0.9	22	3.5	0	....	0	....	0	....	22	3.3
1–1.9	72	2.1	3	3.8	0	....	0	....	75	2.1
2–2.9	0	....	36	2.4	0	....	0	....	36	2.4
3 or more	0	....	4	1.1	131	2.7	133	3.2	268	2.9
Total	95	2.3	43	2.2	131	2.7	133	3.2	402	2.7
All Volumes										
Less than 0.5	77	1.7	7	2.2	0	....	3	7.5	87	1.7
0.5–0.9	160	1.9	33	3.0	3	0.9	0	....	196	2.0
1–1.9	256	1.8	43	1.7	3	0.9	0	....	302	1.8
2–2.9	38	1.9	134	2.4	16	1.7	0	....	188	2.2
3 or more	9	0.6	45	3.1	314	2.7	651	2.9	1,019	2.8
Total	540	1.8	262	2.4	336	2.5	654	2.9	1,792	2.3

The accident rates on four-lane divided curves show a persistent increase with traffic volume. The information is on the skimpy side, but the trend seems definite. The types 2 and 3 rates have statistically significant slopes, with the accident rate increasing by about 0.3 for each additional 1,000 vehicles per day.

For the four-lane divided curves with controlled access, the situation is similar to that of the four-lane divided tangents with controlled access. The apparent decline in the accident rate with increasing volume is mainly due to the difference between Pennsylvania and Connecticut. As in the earlier case, the high volumes are all from Connecticut, while practically all of the traffic below 5,000 vehicles per day is from Pennsylvania.

In summary, on all but the two-lane roads, the accident rate on curves varies with volume in much the same way as on tangents. The general tendency is for higher-than-average volumes to cause higher-than-average accident rates, with some decline in the accident rate at extremely high volumes.

The two-lane curves are different. They show a negative correlation between accident rate and traffic volume throughout the volume range. This is thought to be due to the greater care with which people drive under obviously dangerous conditions.

Two-Lane Curves: Effects of Other Features

Degree, grade, and volume

There are two basically different ways in which curvature and volume together might affect the accident rates. Even if the effects were really independent, there might be intercorrelation between the two factors—i.e., a tendency for the roads having higher-than-average curvature to have either higher-than-average or lower-than-average volume—so that an effect of curvature might appear to be an effect of volume, or vice versa. Multiple regression analysis separates the effects and assigns each to its proper cause.

Or there could be interaction between the two factors—the kind of situation in which the effect of volume at low degrees of curvature is different from its effect at high degrees, and the effect of curvature at low volumes is different from that at high volumes.

Both of these possibilities were investigated. The intercorrelation between degree of curvature and volume is negligible, and the partial correlations between the type 1 accident rate and each of the two factors are both statistically significant, with the same signs as the simple correlations. In other words, for roads of the same curvature the average effect of increased volume is to reduce the accident rate by a significant amount, and for roads carrying the same volume the average effect of increased curvature is to increase the accident rate by a significant amount.

The interaction between curvature and volume can be tested by making a two-way classification of the highway sections by both curvature and volume. This is done in table 20. The type 1 rate shows no particular interaction, since it drops with increasing volume in each curvature group. The types 2 and 3 rates do show interaction, and it is exactly the sort one would expect. At low degrees of curvature—i.e., on the curves which are most like tangents—the hazard increases with volume, just as on the tangent sections. On curves sharper than 6

Table 24.—Accident rates (type 3) on two-lane curves, by volume of traffic, pavement width, and shoulder width

[rates are per million vehicle-miles]

Pavement width, in feet	Number of accidents and accident rate when shoulder width is—									
	Less than 5 feet		5 to 7.9 feet		8 to 9.9 feet		10 feet or more		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day										
16 or less	28	1.5	5	1.1	0	....	0	....	33	1.4
18	257	2.7	153	2.0	4	1.3	1	0.7	415	2.4
20	473	3.0	219	1.9	40	1.4	33	2.2	765	2.4
21–22	58	2.3	5	0.7	2	1.7	26	2.3	91	2.0
23–24	22	2.5	31	1.4	1	0.3	0	....	54	1.6
25 or more	15	1.3	5	1.2	0	....	9	4.2	29	1.6
Total	853	2.7	418	1.8	47	1.3	69	2.3	1,387	2.3
5,000–9,900 Vehicles per Day										
16 or less	1	0.9	1	0.7	0	....	0	....	2	0.8
18	7	3.5	12	1.2	5	5.0	0	....	24	1.8
20	230	3.2	84	2.4	3	1.2	10	2.0	327	2.9
21–22	6	2.7	13	2.3	0	....	3	3.3	22	2.5
23–24	0	....	2	.6	1	3.3	0	....	3	.8
25 or more	9	4.7	8	2.0	0	....	7	2.8	24	2.9
Total	253	3 2	120	2.0	9	2.4	20	2.4	402	2.7
All Volumes										
16 or less	29	1.5	6	1.0	0	....	0	....	35	1.4
18	264	2.7	165	1.9	9	2.2	1	0.7	439	2.3
20	705	3.1	305	2.0	43	1.4	43	2.1	1,096	2.5
21–22	64	2.3	18	1.4	2	1.7	29	2.4	113	2.1
23–24	22	2.5	33	1.3	2	0.6	0	....	57	1.5
25 or more	24	1.7	13	1.6	0	....	16	3.6	53	2.0
Total	1,108	2.8	540	1.8	56	1.4	89	2.3	1,793	2.3

degrees it is the other way around; here the accident rate is lower when there is more traffic volume.

Another way of looking at this interaction is in terms of the effect of changing the curvature at different fixed volume levels. This effect is the same for all three types of rates. At volumes below 5,000 vehicles per day, the accident rate rises steadily with increasing curvature. At volumes over 5,000 the accident rate is lower on sharp curves than on moderate curves.

These facts strengthen the belief that traffic volume affects the accident rate differently on two-lane curves from the way it does on two-lane tangents.

Table 21 gives the type 3 accident rates by degree of curvature and percentage of grade for two-lane curves carrying various ranges of traffic volume. At low volumes the accident rate goes up with increasing curvature, while the effect of grade is not statistically significant. At higher volumes it is not the curvature but the grade that matters; steeper grades make the accident rates larger. This peculiar pattern occurs with both the type 1 and the type 3 rates (the type 2 rates were not computed). In the 0 to 4,900 volume group the partial correlation with curvature is the only significant one, while in the 5,000 to 9,900 volume group the partial correlation with grade is the only significant one.

Curve frequency and other items

Figure 6 shows the accident rates on two-lane curves (of all degrees) as a function of curve frequency. The type 1 rate is highest when curves are very rare, suggesting that a curve is most hazardous when it is unexpected. The types 2 and 3 accident rates have their high values when there are more than five curves per mile.

Table 22 separates the figures into several ranges of curvature. At first glance the most prominent feature of this table is that the types 1 and 3 accident rates have their greatest values for the highest curvature and the lowest frequency. But closer study of the supporting facts shows that this is due to five curves in Iowa which contribute 11 out of the 31 accidents in this group. The other 20 accidents in the group occurred at rates similar to those for other curvatures and frequencies.

The table does not suggest any general conclusions about the effect of curve frequency on the accident rates. The degree of curvature, as before, is positively correlated with the accident rates.

Table 23 gives the accident rates on two-lane curves by curve frequency and sight-restriction frequency for various volume groups. There is a high intercorrelation between the two frequencies, so that while high values of both frequencies cause higher accident rates than low values of both frequencies, it is impossible to tell which of the two frequencies is responsible. (In statistician's language, the multiple correlation is statistically significant, but the partial correlations are not.)

Table 25.—Accident rates (type 3) on two-lane curves, by volume of traffic, percentage of commercial traffic, and percentage of night traffic

[rates are per million vehicle-miles]

Commercial traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is night traffic is—							
	0 to 19 percent		20 to 29 percent		30 to 39 percent		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day								
0–9.9	0	....	0	....	83	3.0	83	3.0
10–14.9	297	3.1	79	2.3	188	2.6	564	2.8
15–19.9	42	2.9	138	1.9	198	2.5	378	2.2
20–24.9	1	1.0	105	1.5	164	1.6	270	1.5
25 or more	0	....	71	2.9	21	2.0	92	2.6
Total	340	3.0	393	1.9	654	2.2	1,387	2.3
5,000–9,900 Vehicles per Day								
0–9.9	0	....	0	....	38	3.0	38	3.0
10–14.9	88	3.7	11	1.5	92	2.4	191	2.7
15–19.9	18	3.5	0	....	64	2.1	82	2.1
20–24.9	0	....	3	2.5	83	2.9	86	2.9
25 or more	0	....	5	2.9	0	....	5	2.9
Total	106	3.6	19	1.8	277	2.5	402	2.7
All Volumes								
0–9.9	0	....	0	....	121	3.0	121	3.0
10–14.9	385	3.2	90	2.2	280	2.5	755	2.7
15–19.9	60	3.1	138	1.8	264	2.3	462	2.2
20–24.9	1	1.0	108	1.5	249	1.9	358	1.7
25 or more	0	....	76	2.9	21	2.0	97	2.8
Total	446	3.1	412	1.9	935	2.3	1,793	2.3

Table 26.—Accident rates at intersections at grade[1] on two- and three-lane roads, by total volume of traffic and percentage of cross traffic

[rates are per 10 million vehicles]

Cross traffic (percent of total)	Accidents on two-lane roads						Accidents on three-lane roads					
	0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day		0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Type 1 Accident Rates (All States, Adjusted)												
0–9	678	3.6	229	2.2	9	0.9	17	4.1	25	9.6	24	34.4
10–19	116	11.3	56	6.3	0	....	0	....	0	....	0	....
20 or more	162	9.2	118	8.7	3	1.9	0	....	30	40.8	0	....
Type 2 Accident Rates (Selected States, without Adjustment)												
0–9	363	2.0	213	1.8	9	0.8	2	3.3	6	2.1	24	26.7
10–19	63	7.1	54	5.5	0	....	0	....	0	....	0	....
20 or more	118	6.9	117	8.1	3	1.9	0	....	30	25.0	0	....
Type 3 Accident Rates (All States, without Adjustment)												
0–9	678	2.0	229	1.8	9	0.8	1	1.7	25	3.5	24	26.7
10–19	116	6.0	56	5.3	0	....	0	....	0	....	0	....
20 or more	162	6.5	118	7.5	3	1.9	0	....	30	25.0	0	....

[1] Excluding rotary intersections.

Table 27.—Accident rates at intersections at grade[1] on four-lane roads, by total volume of traffic and percentage of cross traffic

[rates are per 10 million vehicles]

Cross traffic (percent of total)	Accidents on undivided roads						Accidents on divided roads[2]					
	0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day		0 to 4,900 vehicles per day		5,000 to 9,900 vehicles per day		10,000 or more vehicles per day	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Type 1 Accident Rates (All States, Adjusted)												
0–9	33	8.8	184	9.9	302	4.6	15	3.6	131	5.4	309	7.8
10–19	15	22.9	22	48.7	123	32.3	7	17.5	25	10.8	130	28.3
20 or more	0	....	34	56.7	236	42.2	0	....	13	13.6	97	22.4
Type 2 Accident Rates (Selected States, without Adjustment)												
0–9	13	4.6	62	3.8	255	3.1	15	3.3	91	3.4	238	4.5
10–19	14	28.0	8	8.0	123	28.6	7	17.5	25	11.4	130	24.5
20 or more	0	....	15	21.4	227	36.0	0	....	13	11.8	97	19.4
Type 3 Accident Rates (All States, without Adjustment)												
0–9	33	3.4	184	3.5	302	3.0	15	3.2	131	3.2	309	4.2
10–19	15	21.4	22	14.7	123	28.6	7	17.5	25	10.0	130	24.5
20 or more	0	....	34	18.9	236	35.2	0	....	13	11.8	97	19.4

[1] Excluding rotary intersections. [2] Excluding those with controlled access.

Since the preceding section seemed to indicate that curve frequency did not have any consistent effect, it can only be concluded that the effect of curve frequency is not very clear.

A breakdown of the effects of pavement width, shoulder width, and traffic volume is given in table 24. On two-lane curves the effect of pavement width is too irregular to be statistically significant, but 24-foot wide sections are consistently safer than 20-foot sections. The partial correlation with shoulder width is significant; the accident rate goes down by about 0.15, on the average, for each additional foot of shoulder. These results differ from the situation on two-lane tangents, where neither pavement width nor shoulder width has a consistent effect on the accident rate.

Table 25 shows the effects, on two-lane curves, of commercial traffic and night traffic. Neither effect is statistically significant.

In summary, the effect of volume in reducing the accident rate on two-lane curves seems to be stronger for sharp curves than for flat curves. This is logical, for one would expect flat curves to be intermediate, in their accident potential, between sharp curves and tangents.

The frequency of curves does not appear to have any consistent effect on the accident rate, even when the curves are subdivided by degree of curvature. The frequency of sight restrictions has a similarly uncertain effect.

Wide shoulders definitely help to reduce the accident rate on two-lane curves, and 24-foot pavements are consistently safer than 20-foot pavements. This is in contrast to the situation on two-lane tangents, where shoulder width has no particular effect.

The effect of grade is peculiar. At low traffic volumes there is no particular effect, but on roads carrying more than 5,000 ve-

Table 28—Accident rates (type 3) at intersections at grade[1] on two-lane roads, by total volume of traffic, percentage of cross traffic, type of intersection, and percentage of night traffic

[rates are per 10 million vehicles]

Night traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is cross traffic is—											
	0 to 9 percent						10 percent or more					
	Three-way intersection		Four-way intersection		Total		Three-way intersection		Four-way intersection		Total	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day												
10–19	83	2.9	48	5.0	131	3.4	18	15.0	30	23.1	48	19.2
20–29	143	2.0	185	3.9	328	2.8	58	5.4	67	7.1	125	6.2
30–39	112	0.9	107	1.8	219	1.2	53	4.5	52	5.3	105	4.9
Total	338	1.5	340	2.9	678	2.0	129	5.4	149	7.2	278	6.3
5,000–9,900 Vehicles per Day												
10–19	33	2.8	39	5.2	72	3.7	43	13.4	18	13.8	61	13.6
20–29	35	3.3	19	5.0	54	3.8	9	11.2	23	14.4	32	13.3
30–39	69	0.9	31	2.2	100	1.1	58	4.3	37	6.1	95	4.8
Total	137	1.3	89	3.5	226	1.8	110	6.3	78	8.7	188	7.1
All Volumes												
10–19	118	2.8	89	5.1	207	3.5	61	13.9	48	18.5	109	15.6
20–29	178	2.2	204	4.0	382	2.9	67	5.8	90	8.1	157	6.9
30–39	186	0.9	138	1.9	324	1.1	112	4.3	89	5.4	201	4.7
Total	482	1.4	431	3.0	913	1.9	240	5.7	227	7.5	467	6.5

[1] Excluding rotary intersections, interchanges at grade separations, and intersections with more than four approaches.

hicles per day the accident rates are higher on roads with steep grades.

Finally, the percentages of commercial and night traffic have no recognizable effect on the accident rates on two-lane curves.

Intersections

The accident rates at intersections are computed somewhat differently than on tangents and curves, since the length of an intersection is not particularly relevant to its accident potential. The base which has been used instead of vehicle-mileage is the total number of vehicles using the intersection. To keep the numbers at a manageable size, the accident rates have been expressed in terms of the number of accidents per 10 million vehicles. The only intersections analyzed have been intersections at grade, excluding rotaries, and the number of lanes at the intersections are those along the roads included in the study, irrespective of the number of lanes on the side or cross road. By volume is meant the total number of vehicles entering the intersection from all approaches; the percentage of cross traffic is the percentage of this total volume which enters the intersection from roads other than the study route.

Table 26 presents the accident rates for two-lane roads at these intersections, for various traffic volumes and percentages of cross traffic. The percentage of cross traffic is of crucial importance. At every volume level where there is an adequate sample, the accident rate is more than twice as high when the cross traffic exceeds 10 percent of the total as when the cross traffic is less than 10 percent. Additional increases in cross traffic to 20 percent or more do not cause appreciable further increases in the accident rates.

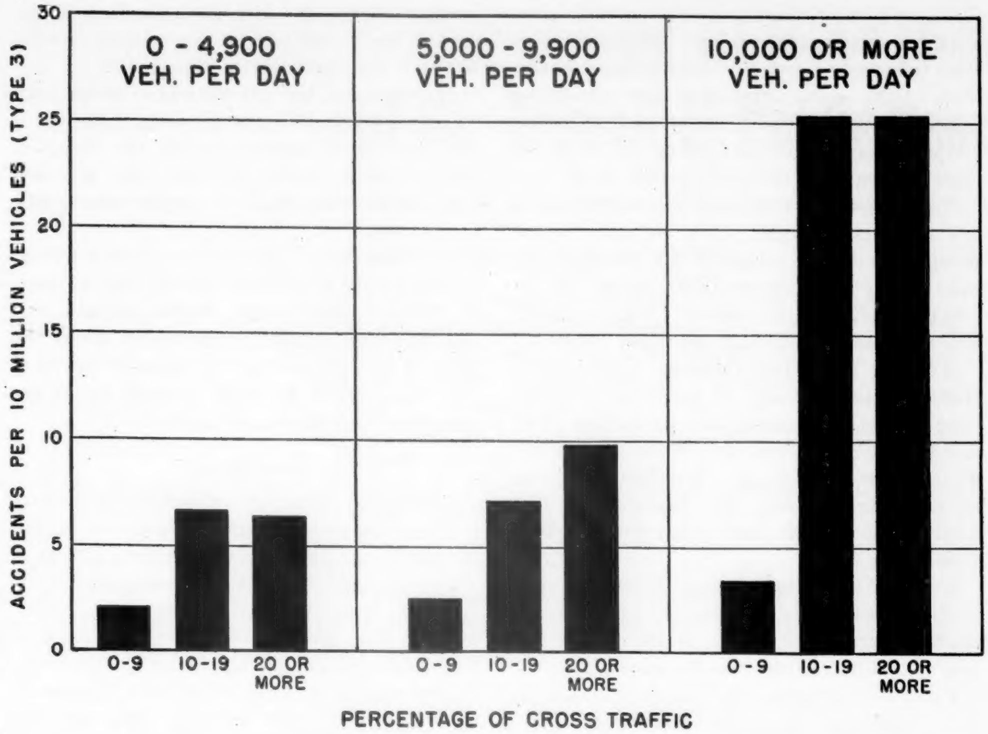

Figure 7.—Accident rates at intersections (all roadway types), by total volume and percentage of cross traffic.

Table 29.—Accident rates (type 3) at intersections at grade[1] on two-lane roads by total volume of traffic, percentage of cross traffic, and percentage of commercial traffic

[rates are per 10 million vehicles]

Commercial traffic (percent of total)	Number of accidents and accident rate when percentage of total traffic that is cross traffic is—			
	0 to 9 percent		10 percent or more	
	Number	Rate	Number	Rate
Traffic Volume 0 to 4,900 Vehicles per Day				
5–9.9	11	0.6	8	3.3
10–14.9	208	2.3	88	6.4
15–19.9	174	1.9	121	6.3
20–24.9	215	1.8	57	6.0
25 or more	80	3.1	16	9.4
Total	688	2.0	290	6.3
5,000–9,900 Vehicles per Day				
5–9.9	6	0.5	23	6.6
10–14.9	128	2.7	83	7.5
15–19.9	65	1.7	64	9.1
20–24.9	21	.8	39	4.6
25 or more	8	1.9	2	2.9
Total	228	1.7	211	6.9
All Volumes				
5–9.9	17	0.5	31	5.3
10–14.9	340	2.4	172	6.7
15–19.9	244	1.8	192	7.0
20–24.9	236	1.6	96	5.2
25 or more	88	2.9	18	7.5
Total	925	1.9	509	6.4

[1] Excluding rotary intersections.

Table 30.—Accident rates at structures on two-lane roads with approach pavements less than 30 feet wide, by relative width of structure roadway and adjoining pavement

[rates are per 10 million vehicles]

Relative width	Number of accidents and accident rate at—			
	Bridges and overpasses		Underpasses	
	Number	Rate	Number	Rate
Type 1 Accident Rates (All States, Adjusted)				
Structure narrower by more than 1 foot	21	9.2	0	......
Structure from 1 foot narrower to 1 foot wider	56	5.8	2	2.9
Structure wider by:				
1.1–3.0 ft	81	7.7	6	6.4
3.1–5.0 ft	87	5.2	2	7.5
5.1–7.0 ft	17	2.3	5	5.6
7.1–9.0 ft	4	.2	0	......
9.1–13.0 ft	14	1.0	9	6.0
13.1–19.0 ft	4	.4	0	......
19.1 or more ft	10	1.6	3	2.5
Type 2 Accident Rates (Selected States, without Adjustment)				
Structure narrower by more than 1 foot	15	5.0	0	......
Structure from 1 foot narrower to 1 foot wider	28	4.7	2	3.3
Structure wider by:				
1.1–3.0 ft	70	4.5	6	6.0
3.1–5.0 ft	61	4.1	1	10.0
5.1–7.0 ft	10	1.3	2	2.5
7.1–9.0 ft	4	.2	0	......
9.1–13.0 ft	7	.5	7	5.0
13.1–19.0 ft	4	.5	0	......
19.1 or more ft	10	1.5	2	1.4
Type 3 Accident Rates (All States, without Adjustment)				
Structure narrower by more than 1 foot	21	5.7	0	......
Structure from 1 foot narrower to 1 foot wider	56	3.6	2	2.9
Structure wider by:				
1.1–3.0 ft	81	4.0	6	5.5
3.1–5.0 ft	87	3.1	2	5.0
5.1–7.0 ft	17	1.3	5	3.1
7.1–9.0 ft	4	.2	0	......
9.1–13.0 ft	14	.6	9	4.5
13.1–19.0 ft	4	.4	0	......
19.1 or more ft	10	1.1	3	1.9

The effect of traffic volume on the accident rate is worth noting. Increased volume reduces the accident rate in most cases. This is the same effect that was noted for two-lane curves. The two effects may possibly be related, since neither of them occurs on roads of more than two lanes.

Table 26 also gives the same information for three-lane roads. The information is meager, but the intersections having less than 10 percent cross traffic appear to be much safer than the others. High volumes are associated with high accident rates.

As with the other roadway types, cross traffic between 10 and 19 percent makes an intersection on a four-lane undivided road much more dangerous than if the cross traffic is below 10 percent. Further increases in cross traffic are less important. The total traffic volume has no consistent effect (see table 27).

On four-lane divided roads without controlled access, the conclusion about cross traffic is again the same. High volume appears to increase the hazard somewhat.

Figure 7 presents the accident rates at intersections for all types of roadways combined. The bars in the chart are of uniform width. In all three volume groups, more than 85 percent of the exposure was at intersections with less than 10 percent cross traffic. The evidence is overwhelming that cross traffic in excess of 10 percent of the total traffic makes an intersection very much more dangerous than if the cross traffic is less than 10 percent.

When all road types are combined, it appears that the accident rate goes up with increases in the total volume. But the exact opposite is the case for intersections on two-lane roads.

Three-way intersections are much safer than four-way intersections on two-lane roads, according to table 28. The angle at which the roads intersect (whether T r Y) does not make any appreciable difference. An increase in the percentage of night traffic reduces the accident rate, as with two-lane tangents.

A breakdown of the effect of commercial traffic, cross traffic, and volume at two-lane intersections is given in table 29. There is some tendency for the accident rate to increase with increasing commercial traffic, but it is not statistically significant.

In summary, the percentage of cross traffic at an intersection is important, and so is the number of approaches to the intersection. Night traffic reduces the accident rate, while the effect of commercial traffic is not clear.

It would be of interest to know how the accident rate is affected by the type of traffic control—stop signs, traffic signals, etc. The question cannot be answered from the data in the present study, because there is too little variety in traffic control at the intersections on the study routes.

Structures

Structures have been classified according to the relative width of the roadway at the structure—on the bridge or in the underpass—as compared with the adjoining pavement. Table 30 presents the accident rates on this basis. The table is restricted to two-lane roads with pavements less than 30 feet wide. Extra width in relation to the approach pavement definitely reduces the accident hazard on bridges. Table 31 shows that for bridges having the same *relative* roadway width, the actual width of the bridge pavement also contributes to the safety of the bridge.

There were not enough underpasses in the study to warrant any conclusions about the effect of roadway width in them. Such evidence as there is, based on a total of 28 accidents, indicates that underpasses are considerably more dangerous than overpasses, even though the average extra width of the underpasses in this study is about 2 feet more than that of the bridges.

Conclusions

The conclusions fall under two headings: (1) a brief summary of the findings, (2) a critique of the study itself. The latter is as important as the former in guiding future research into the cause of accidents.

A summary of the findings was given near the beginning of this report and will not be repeated here. A number of significant relations were discovered, while others that had been expected to be clear-cut did not turn out as expected.

At the beginning of the analysis, the question was raised as to which type of accident rate would prove most reliable of the three types used: (1) adjusted accidents (based on total-to-fatal ratio), all States; (2) actual accidents, selected States (selected for their presumed completeness of reporting); (3) actual accidents, all States. The type 3 rate makes the best showing. Of the first 32 analyses to be made (e.g., two-lane tangents by grade, two-lane tangents by volume, etc.), the type 3 rate behaves credibly in 30 of them and is not seriously misleading in any, though it fails to bring out the effect of traffic volume on two-lane curves. The type 1 rate, in contrast, is reasonable in only 25 of the analyses, and in 2 cases gives results which are significant by statistical tests and yet are seriously misleading. The type 2 rate is misleading in only 1 case, but there are 9 cases in which it is excessively irregular or else contains too little data to give useful results. It would seem that, with the present data at least, one can probably do no better than simply to use all the reported accidents at face value.

The most striking feature of the study is the amount of irregularity in most of the results. Few of the data which have been presented in tables and graphs can be fitted by really smooth curves. There is considerable scatter about the overall trends, and it is likely that some subtle relations are masked by these irregularities.

The fluctuations are much larger than one would expect from considerations of the theory of sampling. They may be due, in part, to errors in the data, such as the failure of the original accident reports to specify the accident locations with sufficient accuracy.

But their principal cause is probably the tremendous complexity of the problem itself. Accidents are associated with so many factors, in such a multitude of combinations, that one has to resort to drastic oversimplifications in order to make any kind of order out of a chaotic mass of material. The remarkable thing is not the irregularities in the tabulations but the number of useful conclusions which do emerge. Most of these conclusions were suspected before the present study was begun, but the statistical analyses give them a broader foundation in hard facts than they ever had before.

However, the foundation is not as broad as it ought to be. Too many of the States were unable to provide information of sufficient accuracy and detail for use in this study. While there has been some improvement in this regard since 1941, the situation is still far from satisfactory. If further progress is to be made in understanding the causes of accidents, many of our States will have to make substantial improvements in the quality of their accident reporting.

Table 31.—Accident rates (type 3) at bridges and overpasses on two-lane roads less than 30 feet wide, by relative width of bridge roadway and adjoining pavement, and actual width of bridge roadway

[*rates are per 10 million vehicles*]

Relative width	Number of accidents and accident rate when width of roadway on bridge is—									
	Less than 20 feet		20 to 24 feet		25 to 29 feet		30 to 34 feet		35 feet or more	
	Number	Rate	Number	Rate	Number	Rate	Number	Rate	Number	Rate
Bridge narrower by more than 1 foot	17	8.1	4	2.5	0	....	0	....	0	....
From 1 foot narrower to 1 foot wider	28	5.6	27	2.7	1	0.9	0	....	0	....
Bridge wider by:										
1.1–3.0 ft	5	25.0	76	4.0	0	....	0	....	0	....
3.1–5.0 ft	2	20.0	76	3.1	7	2.6	2	4.0	0	....
5.1–7.0 ft	0	....	13	1.6	2	.5	2	2.0	0	....
More than 7 ft	0	....	0	....	4	.2	14	.7	14	0.9
Total	52	7.2	196	3.1	14	0.4	18	0.8	14	0.9

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OFFICIAL BUSINESS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1953

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$13,014	$20,032	$10,333	375.1	$5,793	$2,903	190.8	$34,504	$17,597	466.9	$60,329	$30,833	1,032.8
Arizona	1,101	6,734	4,756	147.7	2,348	1,672	27.4	5,850	3,517	69.4	14,932	9,945	244.5
Arkansas	7,637	9,484	5,060	328.4	5,281	2,662	180.8	14,162	7,211	233.1	28,927	14,933	742.3
California	4,162	32,930	15,910	173.6	18,922	9,867	76.8	92,996	45,370	225.6	144,848	71,147	476.0
Colorado	3,953	11,637	6,595	278.9	2,040	1,131	55.8	11,694	5,856	144.6	25,371	13,582	479.3
Connecticut	7,577	1,023	565	2.4	3,306	1,626	9.2	12,245	6,234	27.2	16,574	8,425	38.8
Delaware	3,563	650	325	.8				7,257	3,671	34.1	7,907	3,996	34.9
Florida	5,008	22,647	11,487	388.7	5,268	2,743	80.0	16,184	8,181	262.1	44,099	22,411	730.8
Georgia	15,085	10,371	5,382	227.6	3,843	1,921	34.0	37,072	17,787	575.5	51,286	25,090	837.1
Idaho	4,357	12,204	7,439	294.5	2,959	1,856	55.9	8,658	5,540	140.0	23,821	14,835	490.4
Illinois	12,220	44,896	24,043	302.2	33,844	16,966	388.7	53,684	28,219	378.6	132,424	69,228	1,069.5
Indiana	14,117	34,715	18,180	155.9	10,102	4,989	48.7	28,199	15,139	228.1	73,016	38,308	432.7
Iowa	3,719	24,606	13,918	668.4	7,131	3,578	318.5	10,131	5,125	450.7	41,868	22,621	1,437.6
Kansas	6,802	15,632	7,635	1,272.4	8,874	4,432	337.9	12,490	6,015	685.5	36,996	18,082	2,295.8
Kentucky	5,839	16,290	8,797	228.9	8,090	4,356	97.6	16,548	8,257	285.0	40,928	21,410	611.5
Louisiana	4,206	16,878	8,467	132.0	10,701	5,323	62.7	22,812	10,920	126.1	50,391	24,710	320.8
Maine	4,459	3,179	1,737	19.2	3,432	1,903	5.4	11,058	5,144	81.2	17,669	8,784	105.8
Maryland	[illegible],126	9,552	4,466	82.4	2,477	1,229	25.7	9,887	5,328	42.0	21,916	11,023	150.1
Massachusetts	11,823	3,602	1,946	12.2	858	429		43,518	20,723	46.5	47,978	23,098	58.7
Michigan	11,365	26,377	13,409	524.2	10,465	5,250	185.3	58,070	25,394	251.5	94,912	44,053	961.0
Minnesota	6,298	18,841	10,074	1,350.3	9,642	5,088	484.2	9,635	5,298	209.9	38,118	20,460	2,044.4
Mississippi	3,421	17,736	8,929	682.4	8,710	4,486	276.9	15,918	8,356	467.9	42,364	21,771	1,427.2
Missouri	12,253	18,115	9,471	817.7	23,903	11,381	200.6	38,396	20,189	447.9	80,414	41,041	1,466.2
Montana	7,620	14,886	9,369	317.9	3,881	2,254	76.1	15,696	9,455	243.4	34,463	21,078	637.4
Nebraska	16,796	10,893	5,951	495.7	2,283	1,696	63.3	11,830	6,005	289.7	25,006	13,652	848.7
Nevada	5,266	4,935	4,131	114.2	2,211	1,812	50.9	5,018	3,862	138.1	12,164	9,805	303.2
New Hampshire	2,827	4,450	2,225	26.4	85	47		5,892	3,069	37.1	10,427	5,341	63.5
New Jersey	7,580	7,242	3,468	36.7	7,741	3,838	5.2	31,923	15,487	35.3	46,906	22,793	77.2
New Mexico	1,675	6,178	3,944	167.3	4,078	2,593	81.1	7,509	4,776	246.3	17,765	11,313	494.7
New York	42,527	81,047	43,286	93.7	27,755	13,843	28.0	113,999	52,805	325.4	222,801	109,934	447.1
North Carolina	6,551	27,834	13,372	635.1	4,575	2,178	169.3	30,690	14,778	499.6	63,099	30,328	1,304.0
North Dakota	2,104	11,398	5,710	1,349.7	6,473	3,377	463.5	5,848	2,961	441.4	23,719	12,048	2,254.6
Ohio	13,535	34,723	16,823	164.8	8,804	4,357	40.8	85,764	42,955	116.7	129,291	64,135	322.3
Oklahoma	13,181	10,156	5,541	168.4	2,562	1,365	47.5	18,011	9,462	288.2	30,729	16,368	504.1
Oregon	3,809	2,285	1,320	58.3	3,186	1,899	41.3	13,766	8,281	195.0	19,237	11,500	294.6
Pennsylvania	14,713	35,927	15,972	97.7	24,233	12,079	90.0	83,658	41,623	176.7	143,818	69,674	364.4
Rhode Island	1,531	3,907	1,953	39.9	1,053	526	3.6	20,049	10,394	28.2	25,009	12,873	71.7
South Carolina	6,862	10,042	5,352	261.9	3,374	1,692	101.6	16,598	8,564	420.2	30,014	15,608	783.7
South Dakota	1,508	11,688	6,885	783.2	4,234	2,403	234.3	7,415	4,572	390.4	23,337	13,860	1,407.9
Tennessee	9,934	13,715	6,670	393.6	2,730	1,371	83.6	36,187	16,269	321.0	52,632	24,310	798.2
Texas	20,595	11,589	6,278	192.0	12,481	6,634	231.0	60,342	32,623	1,247.4	84,412	45,535	1,670.4
Utah	454	6,170	4,713	107.4	3,583	2,694	42.3	9,275	7,036	138.2	19,028	14,443	287.9
Vermont	1,211	5,088	2,728	49.1	1,176	588	7.1	7,530	3,758	49.4	13,794	7,074	105.6
Virginia	5,394	15,146	7,228	133.8	8,375	4,158	154.1	30,722	14,799	254.7	54,243	26,185	542.6
Washington	2,215	14,288	7,570	241.9	4,163	2,116	79.2	12,998	6,919	70.9	31,449	16,605	392.0
West Virginia	6,092	5,768	2,907	30.4	4,405	2,223	10.1	18,851	9,460	155.7	29,024	14,590	196.2
Wisconsin	5,626	16,994	9,093	318.5	15,206	7,432	242.6	30,244	15,559	297.4	62,444	32,084	858.5
Wyoming	768	5,418	3,529	159.8	1,243	824	43.5	7,734	5,124	137.3	14,395	9,477	340.6
Hawaii	2,094	3,592	1,744	8.6	742	371	3.2	9,328	4,521	19.2	13,662	6,636	31.0
District of Columbia	1,096	12,930	5,854	8.3	4,211	2,104	1.2	7,143	3,281	.7	24,284	11,239	10.2
Puerto Rico	5,664	8,762	4,282	60.0	1,278	608	6.5	12,218	5,807	42.8	22,258	10,697	109.3
TOTAL	380,333	775,182	406,822	14,980.2	354,110	182,873	5,543.8	1,287,206	649,276	12,485.8	2,416,498	1,238,971	33,009.8